Grand Coulee Dam. Total capacity 1,944,000 kw. (Courtesy of U. S. Bureau of Reclamation.) *Frontispiece*

HYDROELECTRIC HANDBOOK

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With the Assistance of Contributors

SECOND EDITION

NEW YORK · JOHN WILEY & SONS, INC.
LONDON · CHAPMAN & HALL, LIMITED

PRINTED IN THE UNITED STATES OF AMERICA

PREFACE TO SECOND EDITION

Since the first edition of this book was published there has been an enormous advance and accumulation of essential data pertaining to the design of hydroelectric power development. In fact, the progress of the art has been so rapid that it was found necessary to rewrite most of the text of the first edition.

A complete exposition of designing methods of all the structures necessary in hydroelectric power developments cannot be contained in a book of this size.

Sufficient information is therefore presented to give the reader an accurate picture of the problems involved and to provide sufficient data for preliminary designs, estimates of cost, and reports, but it will be necessary for the designer to consult treatises on the complete design of individual structures and appurtenances thereof when the final design of the development is to be made. Information contained in *Engineering for Dams* by Creager, Justin, and Hinds (John Wiley & Sons, 1945) has been drawn upon freely in chapters on dams.

For the second edition the authors are indebted to the listed contributors of chapters and sections and to many other engineers, some mentioned in the text, who have contributed valuable data. Others to whom the authors desire to acknowledge indebtedness are Messrs. Neville C. Courtney, Stephen H. Haybrook, Clyde W. Hulburd, Joel B. Justin, Richard A. Lane, Byron O. McCoy, and the late Carl Ashley for engineering calculations and editorial work, and Mr. Adolph F. Meyer for suggestions on evaporation.

William P. Creager
Joel D. Justin

Philadelphia, Pa.
December, 1949

PREFACE TO FIRST EDITION

The aim of this book is to present a compendium of all phases of modern hydroelectric practice. It is designed to contain sufficient descriptive matter to make it valuable to the student and also to include much that is new in theory and practice and thus to be of considerable interest to the practicing engineer.

Although the preparation of the book required several years of work, the last few months before going to press were spent in bringing the manuscript up to date. It is therefore believed that the text is truly representative of modern practice.

The editors are indebted to many engineers for helpful advice and contributions to the data contained in this book. Most of these persons have been mentioned in the text. Others to whom the editors desire to acknowledge indebtedness are Messrs. Charles A. Bissell, Albert S. Crane, Percy C. Day, Raymond D. Johnson, Frank C. Kelsey, Adolph F. Meyer and Forrest Nagler.

Much help has been received from the engineers of the U. S. Weather Bureau, the U. S. Geological Survey, and the U. S. Reclamation Service, and from the publications issued by them; also from the members of the special committees of the various national engineering societies, particularly those of the National Electric Light Association, and from their publications.

Special credit is due to Mr. Gardner C. George for checking the text and preparing the index, to Miss Mary Morris for copy editing, and to Miss Dorothy Wood for work on the cut proofs.

William P. Creager
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PRELIMINARY STUDIES AND ECONOMICS

CHAPTER 1

RAINFALL

1. General. As the extent to which a stream should be developed for power depends upon the flow of the stream, as do also many details of design and the amount of energy available, a knowledge of the elements of hydrology is of great importance to the hydraulic engineer.

Of the precipitation that falls upon a watershed, a part is lost by evaporation, both by transpiration through the pores of vegetable surfaces and by direct evaporation from land and water surfaces; an inconsequential amount is used by plants in the formation of vegetable tissue; a part may be lost through deep seepage or diversion out of the watershed; and the remainder appears as runoff at the site of proposed use. A portion of this remainder, or all of it, may be developed for power.

"Rainfall," when used in a general sense, or "precipitation," may be defined as the total condensation of moisture from the atmosphere that reaches the earth, including all forms of rain, ice, and snow. The rate of rainfall is expressed in inches of water during a given period of time.

There is available only meager information concerning the influences affecting the variation of rainfall as regards either time or locality. The variation of rainfall with the seasons of the year generally follows a well-defined rule for each locality, but the influences affecting the variation for different years are too numerous for accurate coordination.

Figure 1 indicates the general variation of mean annual rainfall throughout the United States. In general, the annual rainfall is influenced by topography, altitude, and the proximity of the source of moisture supply.

When moisture-laden air reaches mountainous regions it is deflected upwards. Encountering the lower pressure of the higher atmosphere, it expands, cools, and precipitates its moisture. Even on slight slopes facing moisture-laden winds, precipitation generally increases with the altitude, other things being equal.

2. Rainfall Records. Most rainfall records are obtained by periodic observation of gages. The usual interval is 24 hours. Such observations give little information as to rate of rainfall If the record shows that a certain station had rain of 12 in. during some given day, often all that is known is that there was a total of 12 in. of rain which fell during that 24-hour period. In hydrological studies (particularly in connection with flood studies) it may be of the utmost importance to know the rate of rainfall at

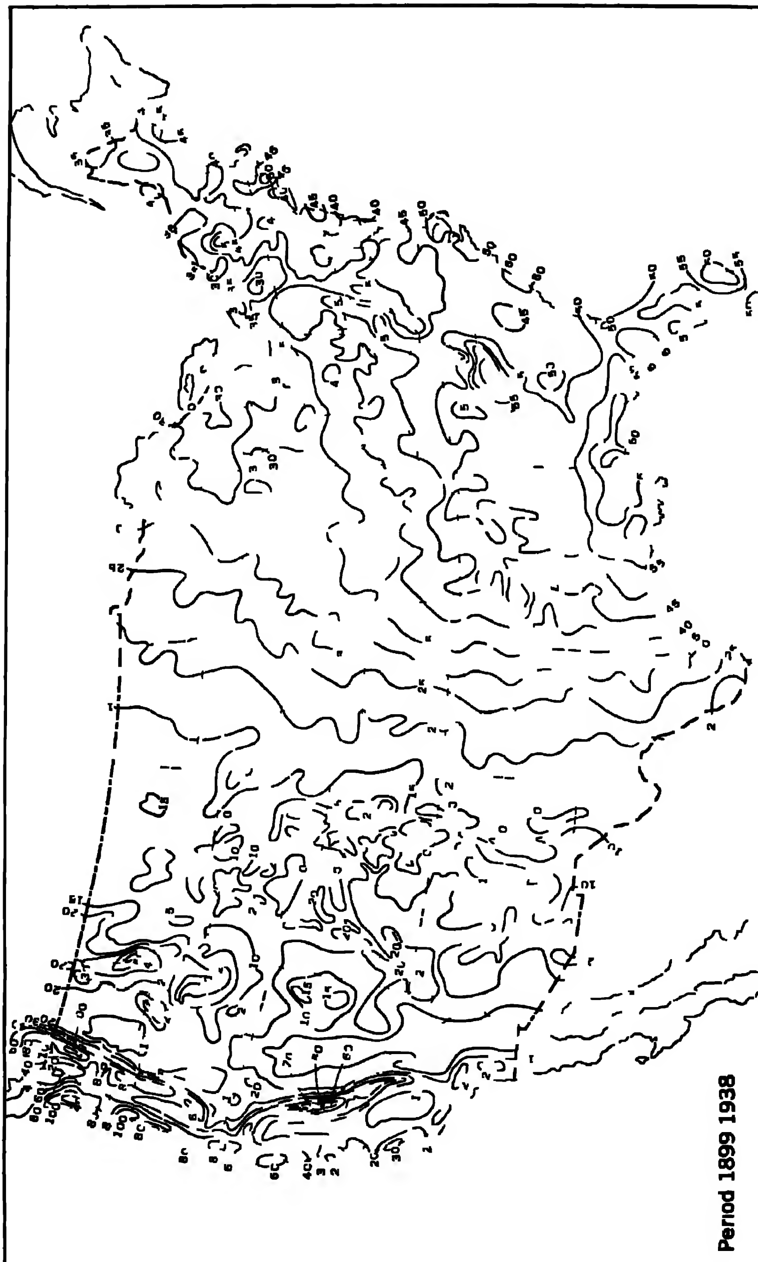

FIG 1 Average annual precipitation (inches) in the United States Page 771 of 1941 Year Book of Agriculture *Climate and Man* House Document No 27 77th Congress 1st session)

any given time. Automatic recording rainfall gages provide this valuable information, and in recent years their use has been increasing.

The United States has been divided by the U. S. Weather Bureau into 106 climatological sections, and monthly bulletins for each section are issued by the Bureau, giving the daily and mean monthly precipitation for each gaging station in the section. Annual bulletins for each section are also published, and these tabulate the monthly and yearly means at each station for all years of record. The bulletins contain data on temperature, wind, and humidity, as well.

All precipitation records must be used with a full realization of their limitations, which may be summarized as follows:

(*a*) Most rainfall stations have been located at or near centers of population, which usually occur below the plane of average elevation of the section. Hence, as rainfall varies with the elevation above sea level, records obtained at these stations are not truly indicative of average precipitation.

(*b*) The monthly precipitation at stations only a few miles apart, subject to the same hydrological conditions, often differs as much as 25 per cent, owing to the limited width of the sharply defined path of severe storms, which constitute a large part of the monthly rainfall. This is particularly true during the season of thunderstorms. Few districts may be found where rainfall stations are close enough together to indicate true monthly averages; but, for periods of a year or more, errors are compensating and a reasonable degree of accuracy may be expected.

(*c*) Annual precipitation varies considerably from year to year, and records of considerable length are necessary to indicate accurately the mean rainfall for a given station.

(*d*) Accuracy of rainfall records varies greatly, and for important hydrological studies a careful investigation of accuracy should be conducted. For instance, an observer may, for convenience, move a rain gage so near his house that it is sheltered and does not correctly record the rainfall. Many factors combine to make the record of a particular rain gage almost valueless.

Table 1 gives the maximum percentage of variation from 46-year mean annual rainfall for (1) United States area east of 103rd meridian and (2) 5-state group of West Virginia, Kentucky, Ohio, Indiana, and Illinois.

3. Mean Annual Rainfall on an Area. In estimating the runoff of a stream having no discharge records, the recorded runoff of a neighboring stream is often used, in conjunction with a study, among other things, of the difference in mean annual rainfall on the two areas. Rainfall records on a given area are also used to determine whether the years of river-discharge records for that area represent high or low periods. The following steps are necessary in the compilation and use of data to determine the annual rainfall over a given area or the mean for a series of years.

All annual rainfall records on, or in the vicinity of, the drainage area are tabulated, and missing records for each station are approximated by interpolation between records of adjacent stations, so that the records at each sta-

TABLE 1

PROBABLE DEVIATION OF SHORT-TERM PRECIPITATION RECORDS FROM THE TRUE MEAN *

Duration of Record	(1) U. S. Area East of 103rd Meridian—Probable Percentage of Error, + or −	(2) 5-State Group—Probable Percentage of Error, + or −
2	12.8	20.2
3	10.3	14.1
4	8 5	11 5
5	7 5	8.1
10	3.1	5.6
15	1.8	4.0
20	1.2	2.6
25	1.5	3.0
30	1 4	1.3
35	0.7	1.0
40	0.4	0.9

* Used also to indicate probable deviation of annual runoff.

tion cover the same period of years. The mean annual rainfall at each station is then determined.

The use of an isohyetal map furnishes the most accurate method of determining the mean annual rainfall for a given area. An isohyetal map consists of a series of lines of equal rainfall, indicating the variation in mean annual rainfall in a manner similar to that in which the ordinary topographical maps indicate the variations in ground-surface elevations by means of contours. An isohyetal map, showing mean annual precipitation in the United States, is reproduced in Fig. 1. Isohyetals covering smaller areas would indicate variations in rainfall which it would be impossible to include on the small scale of Fig. 1. The more numerous the rainfall stations between which the isohyetals are plotted, the greater the accuracy of the map.

The maps should be drawn by plotting all rainfall stations, with their mean annual rainfall, on a topographical map of adequate scale. The isohyetals should then be drawn, due consideration being given to the effect of the topographical features on probable rainfall, because a straight-line variation of rainfall between stations does not always obtain, particularly if the two stations are in different valleys. The direction of prevailing winds and the location of the major source of moisture supply should be kept in mind as affecting the variation of precipitation with altitude. The deflection of the prevailing winds by mountain ranges and the larger valleys should be studied in connection with the resulting effect on probable rainfall.

The method of computing the mean annual rainfall, for a given area, from an isohyetal map differs in no way from that used to determine the

average elevation of an area from the contours of an ordinary topographical map.

Isohyetal maps may be used for the mean annual rainfall for the years of record or for each year of record; but, when the rainfall on an area for each year of record is required, the use of isohyetal maps is tedious and other methods are frequently employed, particularly if the available information does not permit of great accuracy.

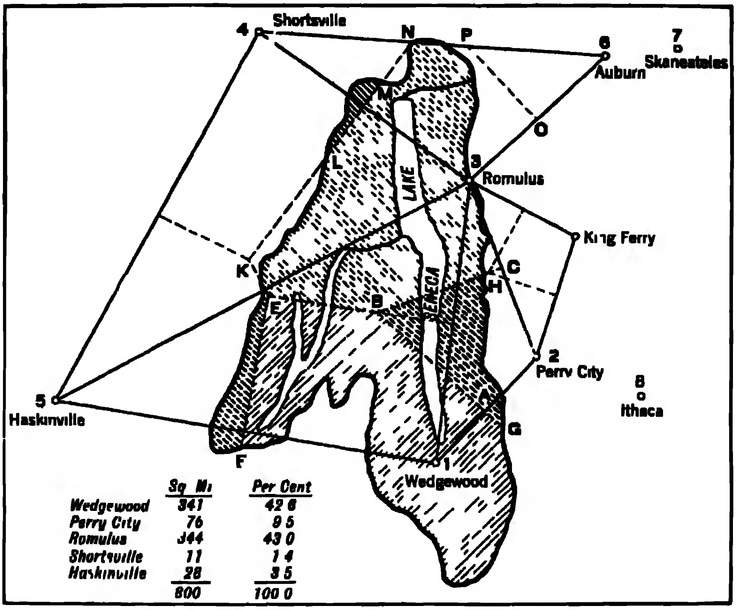

FIG. 2. Weighted method of determining rainfall.

The "weighted method" for determining the rainfall on an area is commonly used for approximate estimates and for cases where a comparison of the rainfall of various years is to be used to extend limited records of river-discharge gagings. In the latter case the relative rainfall for each year of record is as useful as the actual rainfall. The weighted method, though less exact than the isohyetal method, embodies errors which are approximately the same for each year and therefore affects the relative rainfall very little.

In the weighted method, each rainfall station is given a weight depending upon the percentage of the whole area which it is considered to represent. R. E. Horton has devised the following method for determining such weights.

Plot the different rainfall stations within and adjacent to the area on a small-scale outline map of the area, as indicated in Fig. 2. Lines connecting the different stations are drawn, as 1–2, 2–3, and perpendiculars are erected at the median points on these lines, as *AB* and *BC*. Station 1 lies nearer

than any other to all points within the enclosed area *ABEFG*. Station 2 lies nearest to the portion *GABC*, and Station 3 lies nearest to the portion *HBKLMN*. Points on the line *GB* are equidistant from Stations 1 and 2. Stations 6, 7, and 8 are not applicable to any portion of the area and may be eliminated. This is evident for Station 6, since the perpendicular, *OP*, lies wholly outside the area. Hence no part of the area is as near to Station 6 as to Station 3. The graphical construction is easily checked by the fact that the perpendiculars to the median points of the three sides of a triangle intersect at a common point, as *K* for the triangle 3–4–5. The areas of the figures nearest the different stations are determined, and the corresponding proportion of the total area is computed. These proportions are the weights to be given to the records at the corresponding stations.

The precipitation at each station is multiplied by its weight, and the average of the resulting weighted quantities is the probable average rainfall on the area.

As explained heretofore, it may sometimes be definitely determined that, owing to the characteristics of the topography and prevailing winds, a rainfall station will not truly represent the average rainfall on the corresponding area from which it derives its weight. In such circumstances, the recorded rainfall at that station for each year should be increased or decreased as the case may be; or the correction may be applied by using the actual rainfall records but changing the weight a corresponding amount, in which event the sum of the weights would not equal unity.

See also Section 6, Chapter 6.

4. The Rainfall Year. In making comparisons of annual rainfall to annual runoff, it should be remembered that much of the November and December rainfall may run off in January or February of the next year. A comparison of the rainfall of a calendar year with the runoff during that year would, therefore, be liable to considerable error. Since rainfall during the dry season has no appreciable effect on the total annual runoff, more accurate comparisons may be made by using a "runoff year," beginning at the end of the dry season when ground storage is about depleted.

The U. S. Geological Survey has adopted the practice of publishing runoff data in runoff years, beginning with October 1, the end of the low-stream-flow or dry season throughout most of the United States. Comparisons with rainfall should be made for a "rainfall year," covering the same period or, since runoff is not exactly coincident in time with rainfall, for large areas, a month or two ahead, or for the year beginning September 1 or August 1.

5. The Average Frequency of Dry Years. If rainfall records of sufficient duration are available, the average frequency of occurrence of given mean annual rainfall intensities may be determined with a fair degree of accuracy by the law of probabilities as described in Section 4, Chapter 6.

The annual rainfall at Peoria, Peoria Co., Ill., has been thus examined in Table 2.

TABLE 2

COMPUTATIONS FOR DETERMINING THE FREQUENCY OF ANNUAL RAINFALL AT PEORIA, PEORIA CO., ILL. FIFTY-THREE YEARS OF RECORDS

(1) Annual Inches of Rainfall	(2) Number of Occurrences	(3) Summation of Occurrences, n	(4) Percentage of Years, p	(5) Yearly Frequency
24	1	1	0.944	105.8
25	1	2	2.83	35.3
26	2	4	6.60	15.15
27	2	6	10.38	9.65
28	1	7	12.27	8.15
29	3	10	17.92	5.57
30	1	11	19.8	5.05
31	6	17	31.1	3.22
32	2	19	34.9	2.86
33	6	25	46.2	2.16
34	1	26	48.1	2.08
35	2	28	51.9	1.93
36	6	34	63.2	1.58
37	1	35	65.0	1.54
38	1	36	67.0	1.49
39	3	39	72.7	1.37
40	5	44	82.1	1.22
41	1	45	84.0	1.19
42	3	48	89.7	1.11
43	1	49	91.5	1.09
44	1	50	93.5	1.06
45	..	..	...	....
46	..	..	...	....
47	..	..	...	....
48	..	.	....	..
49	1	51	95.3	1.05
50	1	52	97.2	1.03
51	.	.	....	...
52	..	..	...	....
53	..	..	.	....
54	1	53	99.1	1.01
	$m = 53$			

The second column indicates the number of times the mean annual rainfall had a value between the corresponding amount in Col. 1 and that in the line next above. Three times during the period of fifty-three years covered by the records, the annual rainfall was between 41 and 42 in.

Column 3 is a summation of Col. 2 and indicates the number of times during the period covered by the records that the rainfall was less than the

amount indicated in Col. 1. Forty-eight times during the period, the annual rainfall was less than 42 in.

Let n = the summation of occurrences as indicated in Col. 3,
m = the total number of occurrences, in this case = 53,
p = the percentage of years in which the annual rainfall probably will be less than a given amount (Col. 4).

Then, from Eq. 5, Chapter 6,

$$p = 100\left(\frac{n - 0.5}{m}\right)$$

The values in Col. 4 are calculated from the above equation. The result indicates that an annual rainfall less than 42 in. may be expected in 89.7% of all future years.

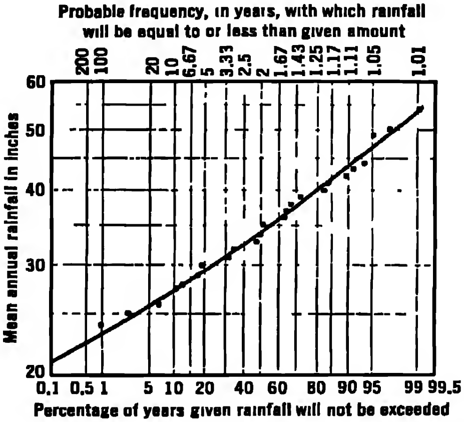

FIG. 3. Probability curve of mean annual rainfall at Peoria, Ill.

Finally, the probable frequency, I, in years, with which the rainfall will be equal to or less than a given amount may be found from Eq. 6, Chapter 6,

$$I = \frac{100y}{mp} = \frac{100}{p}$$

where y is the number of years of record, which in this case equals m, the number of occurrences.

With this equation, Col. 5 may be calculated. The result indicates that a mean annual rainfall less than 42 in. may be expected, on an average, once in 1.11 years. Columns 1, 4, and 5 are plotted in Fig. 3.

The curve projected through the plotted points represents the rainfall probabilities at the station as nearly as they can be determined. It indicates that a mean annual rainfall of less than 23.3 in. should be expected to occur not oftener than once in about 100 years.

This method should give accurate results for records of great length; but most records are for relatively short periods and are subject to the errors pointed out in Section 2. Average yearly rainfall on a drainage area may be used, instead of that of a single station, to obtain corresponding rainfall probabilities for the area.

6. Air Masses and Cold Fronts. By the use of a network of simultaneous airplane observations at 4:00 A.M. Eastern Standard Time, weather permitting, the U. S. Weather Bureau with the aid from the Army and Navy takes daily soundings of the atmosphere, up to 5000 meters. An attached meteorograph automatically records on a chart the pressure, temperature, and relative humidity, from which data the weather may be predicted with some accuracy and the presence of any unusual air mass may be readily discovered and measured.

An air mass is defined as an extensive portion of the atmosphere which approximates horizontal homogeneity as to temperature and moisture content. Such an air mass occurs wherever the atmosphere remains at rest over an area of uniform surface properties for a sufficient time to permit the atmospheric properties to reach approximate equilibrium with respect to the surface beneath. Such a surface is referred to as a "source region."

A polar air mass of cold air originates in the polar regions; a tropical air mass of warm, moisture-laden air originates in the tropics or subtropics. Considering the North American continent as affecting weather and rainfall in the United States, there are the following source regions:

Polar Continental from over Canada, Alaska, and the Arctic.

Polar Pacific from the Arctic or sub-Arctic, reaching the Pacific Coast of Canada and the United States and passing eastward over the continental divide

Polar Atlantic from the North Atlantic Ocean. It rarely affects weather west of the Appalachian Mountains.

Tropical Gulf from the Gulf of Mexico, Caribbean Sea, or farther southeast.

Tropical Atlantic from the Sargasso Sea in the Atlantic Ocean.

The chief sources of rainfall east of the Rocky Mountains are tropical air masses from the Gulf or Southern Atlantic moving northward over the great plains or the costal plain. These warm, moisture-laden air masses are raised and cooled by a general rise in ground elevation by being pushed upwards by heavy, low-lying polar air. As a result their temperature is reduced until they can no longer hold their moisture, and precipitation takes place.

The limiting edge of a polar air mass is commonly called a "cold front." When a tropical air mass, moving northward, encounters a cold front, deposition of moisture occurs, the amount depending upon the extent of the cold front and the humidity and extent of the tropical air mass. Extremely large and moist air masses often result in precipitation of unusual proportions when such a "collision" occurs with a large cold front.

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CHAPTER 2

EVAPORATION

1 General Evaporation in its broadest sense represents practically all of that portion of the rainfall that does not reach the point of ultimate use as stream flow. The process of evaporation consists in the changing of a liquid or solid into a vapor. Transpiration is the vaporization of water from the breathing pores of vegetable matter and, as such, is but a special

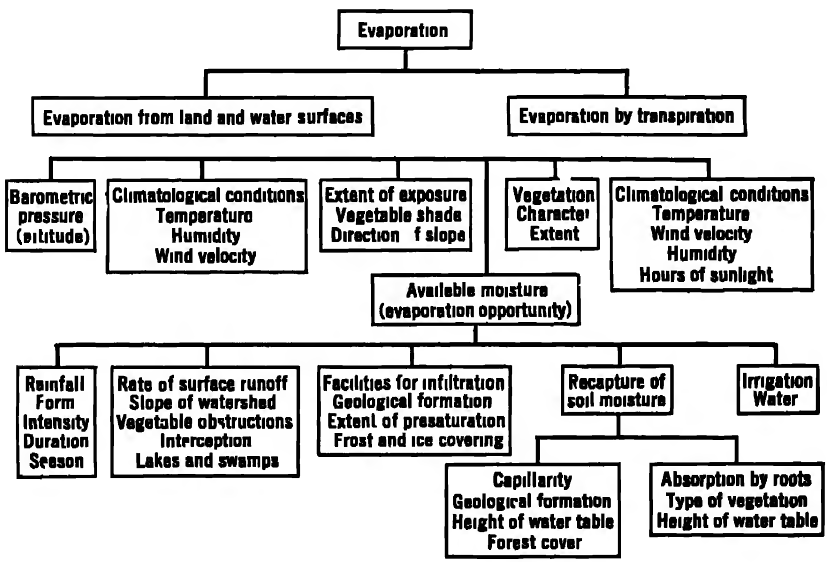

Fig. 1 Chart showing conditions affecting evaporation

form of evaporation. Therefore the term "evaporation," as here used, includes all the rainfall that is returned to the atmosphere from land and water surfaces. For purposes of study, total evaporation has been divided into evaporation from water surfaces and evaporation from land surfaces and transpiration. In Fig. 1 is given a chart showing the conditions affecting total evaporation.

The numerous factors affecting evaporation, the great range in the influence of each and the almost infinite number of possible combinations of those factors make it impossible to compute precisely the distribution and amount of residual rainfall which appears as runoff solely from the rainfall and other physical data commonly available. A thorough knowledge of the factors

that influence evaporation is necessary, however, for the correct interpretation of runoff data and their extension over the period for which rainfall data alone are available. Such knowledge is also helpful in estimating the runoff from drainage areas for which no meter gagings, or only intermittent ones, are available, and for predicting the probable frequency of certain flood discharges of streams.

The rate of evaporation of free moisture from land and water surfaces is controlled by climatological conditions and liquid temperatures. The rate of evaporation of moisture absorbed from the soil and transpired by plants is controlled both by climatological conditions and by the nature and extent of the vegetal cover.

While the *rate* of evaporation at a given time is determined by the factors mentioned, the *amount of water evaporated* during a given interval is governed also by the amount of available moisture present during that interval, or by what Horton termed the "evaporation opportunity." The evaporation opportunity is controlled by the amount, rate, and character of precipitation, the surface runoff, the infiltration, the vegetal cover, and the possibility of recapture of soil moisture by capillarity or through the roots of plants. Water supplied to fields for irrigation offers additional opportunity for evaporation.

2. Evaporation from Water Surfaces. The effect of climatological conditions on evaporation is shown by Fig 2 taken from Meyer's "Evaporation from Lakes and Reservoirs," in *Minnesota Resources Commission Bulletin* [1].* This shows the variation of evaporation from free water surfaces in the United States. The lines of equal evaporation refer to "shallow lakes and reservoirs." The bulletin defines shallow lakes and reservoirs as those not over 35 ft in depth.

It is an established fact that evaporation increases greatly with an increase in temperature and wind velocity, and decreases with an increase in atmospheric humidity. However, evaporation does not cease when the relative humidity reaches 100%. Water vapor is much lighter than the dry gases of the atmosphere and always tends to rise.

During the year, the air temperature, being the effect, follows the heat received from the sun, which is the cause, with considerable lag. The temperature of bodies of water in turn lags materially behind the temperature of the air. This is true for temperature changes during the day and also during the season. The extent of the lag of water temperatures behind air temperatures depends upon the rate of change in air temperature and the depth of the body of water.

Meyer's studies on Lake Superior, reported in his bulletin, indicate that the area of the body of water has relatively little effect upon the rate of evaporation, whereas the depth of the water very materially affects the rate of evaporation.

* Numbers in brackets refer to items in bibliography at end of chapter.

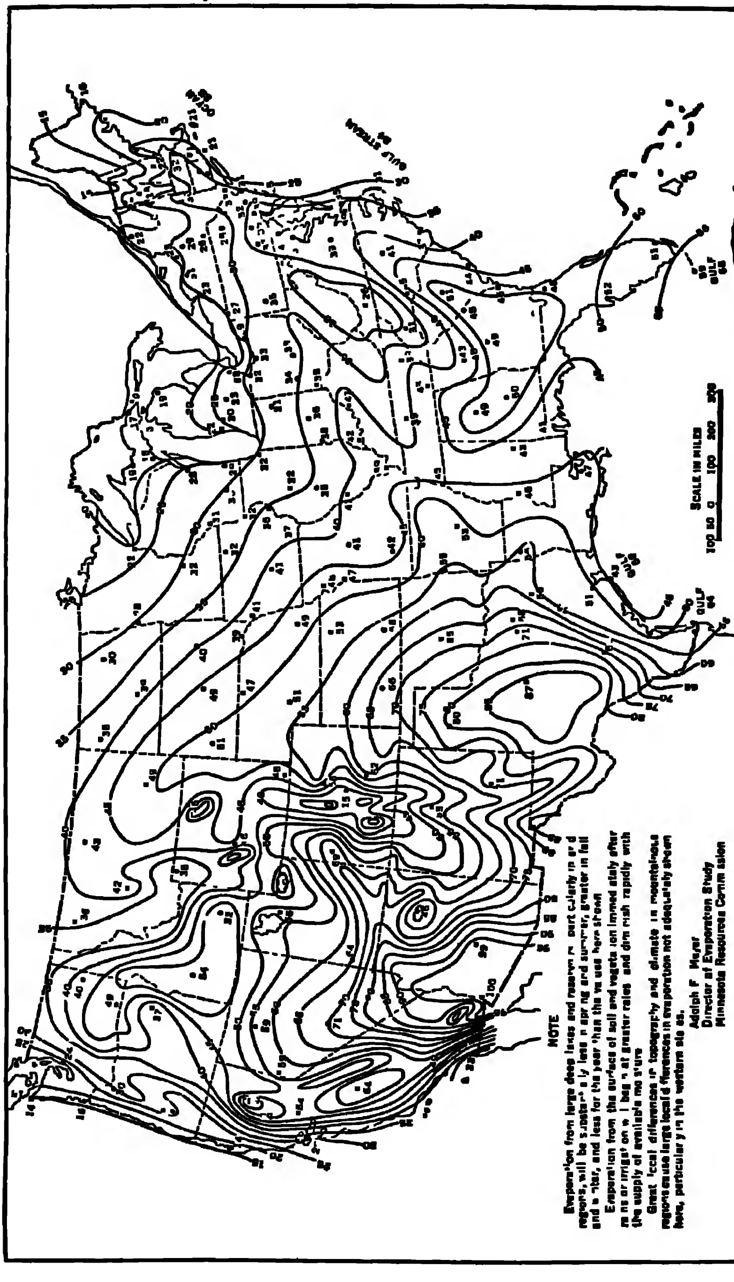

FIG. 2. Mean annual evaporation from shallow lakes and reservoirs: depth in inches.

Although a reduction in barometric pressure is known to cause an increase in evaporation, nevertheless the reduction in temperature which normally accompanies an increase in altitude more than compensates for any increase in evaporation due to the lower barometric pressure at higher altitudes. For example, evaporation on Pikes Peak, with an altitude of more than 14,000 ft, is only about one-fourth as much as at Denver and Pueblo, with altitudes of about 5000 ft.

The monthly variation in evaporation in different parts of the United States is given in Table 1 as compiled by Adolph F. Meyer. These values of

TABLE 1

EVAPORATION FROM LAKES AND RESERVOIRS IN INCHES DEPTH

Station	Jan	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov	Dec.	Annual
Eastport, Me.	0.7	0.7	0.9	1.1	1.4	1.7	2.0	2.1	2.0	1.6	1.1	0.7	16.0
Albany, N. Y.	0.6	0.7	1.1	2.0	3.2	4.3	5.2	4.7	3.4	2.4	1.4	0.8	29.8
Nantucket, Mass.	0.8	0.9	0.9	1.2	1.6	2.2	2.7	2.9	2.7	2.4	1.6	1.0	20.9
Atlantic Ocean off Cape Cod	3.0	2.5	2.0	1.5	1.0	1.5	1.5	2.0	2.5	3.0	3.5	4.0	28.0
Scranton, Pa.	0.6	0.8	1.1	2.0	3.1	4.0	4.7	4.2	3.0	2.3	1.4	0.9	28.1
Richmond, Va.	1.3	1.7	2.2	3.5	4.1	5.0	5.6	4.9	4.1	3.2	2.4	1.5	39.5
Gulf Stream off Cape Hatteras	9.0	9.5	8.5	7.0	5.5	3.5	3.5	3.5	5.5	9.0	9.5	10.0	84.0
Augusta, Ga.	1.6	2.1	3.0	4.2	5.3	6.2	6.5	5.8	5.3	3.9	2.6	1.7	48.2
Miami, Fla	3.0	3.4	4.1	4.9	5.0	4.8	5.3	5.1	4.3	4.1	4.3	2.7	51.0
Gulf Stream off Key West	5.0	5.5	5.5	5.5	5.5	5.5	6.0	6.0	6.0	6.0	6.0	5.5	68.0
Cincinnati, O.	0.8	0.9	1.5	2.9	3.9	5.1	6.2	5.7	4.6	3.4	2.0	1.0	38.0
Nashville, Tenn.	0.9	1.3	1.9	3.3	4.1	5.1	5.8	4.3	4.9	3.7	2.1	1.1	39.6
New Orleans, La.	1.4	2.0	2.6	3.9	4.8	5.5	5.8	5.7	5.7	4.9	3.2	1.7	47.2
Duluth, Minn.	0.3	0.3	0.7	1.4	2.1	2.4	3.7	4.2	3.6	2.4	1.0	0.3	22.4
North Platte, Nebr.	0.8	1.1	2.2	3.7	5.0	6.5	8.6	8.4	6.0	4.6	2.6	1.1	51.5
Columbia, Mo.	0.9	1.1	1.9	3.2	3.7	4.5	6.2	6.3	5.1	4.0	2.5	1.2	40.6
Palestine, Tex	1.6	2.2	3.3	4.4	5.1	6.5	7.8	8.0	6.9	5.3	3.3	1.9	56.3
Brownsville, Tex.	1.5	2.4	3.4	4.4	5.4	6.2	6.8	7.1	5.6	4.6	3.2	1.7	52.3
Havre, Mont	0.5	0.5	1.1	2.5	4.5	6.1	8.2	8.3	5.6	3.3	1.5	0.7	42.8
Yellowstone Park	0.7	0.8	1.2	2.0	3.0	4.4	6.4	6.7	4.8	2.9	1.6	0.7	35.2
Boise, Ida.	0.8	1.2	2.4	3.8	5.3	7.1	10.6	10.1	6.3	3.5	1.8	0.9	53.8
Denver, Colo.	1.6	1.8	2.5	3.7	5.0	7.4	8.8	8.4	6.7	4.6	3.0	1.9	55.4
Pikes Peak, Colo.	0.5	0.4	0.5	0.8	1.3	2.2	2.0	1.8	1.7	1.3	0.9	0.6	14.0
El Paso, Tex.	3.1	4.2	6.3	8.5	10.9	12.4	11.0	9.7	8.4	6.5	4.5	3.1	88.6
Yuma, Ariz.	3.9	4.6	6.5	8.0	9.8	11.5	13.4	12.9	10.7	8.0	6.1	4.5	99.9
Seattle, Wash.	0.8	0.8	1.4	2.1	2.7	3.4	3.9	3.4	2.6	1.6	1.1	0.7	24.5
Baker, Ore.	0.5	0.7	1.4	2.5	3.4	4.4	6.9	7.3	4.9	2.9	1.5	0.6	37.0
Winnemucca, Nev.	0.9	1.3	2.4	3.9	5.3	7.9	11.2	10.8	7.2	4.3	2.5	1.2	58.9
Mt. Tamalpais, Calif.	2.2	2.5	3.0	4.0	5.0	7.5	9.5	9.5	9.0	7.0	4.0	3.0	66.2
San Francisco, Calif.	1.3	1.5	2.1	2.5	2.7	2.9	2.7	2.5	2.9	2.9	2.4	1.7	28.1
Fresno, Calif.	0.8	1.5	2.3	3.0	6.2	9.3	12.0	11.6	8.1	4.6	2.6	1.2	64.1

computed evaporation may be considered representative of actual evaporation from lakes and reservoirs in the regions concerned, within the limits of the present knowledge about factors that influence evaporation. The data

should be quite accurately comparative because the same formula was used at all stations and it was applied to as closely similar meteorological data as could be secured with present instruments and under present methods of observation.

It is often necessary to alter the estimated stream flow to compensate for evaporation from future reservoirs to be built in connection with a water-power development. Usually the area of such artificial bodies of water constitutes a very small percentage of the area of the watershed, and estimates of evaporation based on Meyer's map are well within the degree of precision in the stream measurements upon which such estimates of flow are based. For reservoirs intended to equalize the flow of streams over a long period of years, the cumulative evaporation loss over 10, 15, or 20 years becomes an important consideration and deserves careful attention.

The engineers of the U. S. Bureau of Reclamation prefer to base their estimates of evaporation from large reservoirs upon actual experiments at the site or at the nearest experimental station rather than to use any existing evaporation equation.

It is the opinion of the Climatological Division of the U. S. Weather Bureau that there is no safe method for determining evaporation except by actual pan observation. However, the reduction of all pan observations to equivalent reservoir evaporation is still uncertain, even though a ratio of 70% of U. S. Weather Bureau pan evaporation is considered reasonably indicative of reservoir evaporation in many parts of the United States. In California, Young [7] recently found a coefficient of 78%. In view of the large changes which occur in evaporation even when the average of successive decades is considered, as shown in Meyer's bulletin [1], Table 2, it is evident that the few years of pan observations which can be made prior to the construction of a reservoir may give a far from true picture of either mean annual evaporation or the change in evaporation from year to year. To illustrate, the evaporation from water during the drought of 1934 in the Middle West was about twice the evaporation in 1943.

Many engineers, in estimating the evaporation for a particular reservoir, have assumed that the evaporation is approximately equal to the precipitation in that region. This rule-of-thumb assumption is erroneous since both the precipitation and evaporation differ in different parts of the country, depending on unrelated factors.

Figure 3* serves to illustrate the possible difference between evaporation and precipitation in various parts of the country. At Elkins, W. Va., the chart shows a large excess of precipitation over evaporation; while at Havre, Mont., the evaporation is greatly in excess of precipitation. Also, for comparison, the chart shows precipitation and evaporation approximately equal at Memphis, Tenn.

* Taken from Meyer's bulletin. For similar graphs of other localities, see Ref. 1 of Bibliography, Section 9.

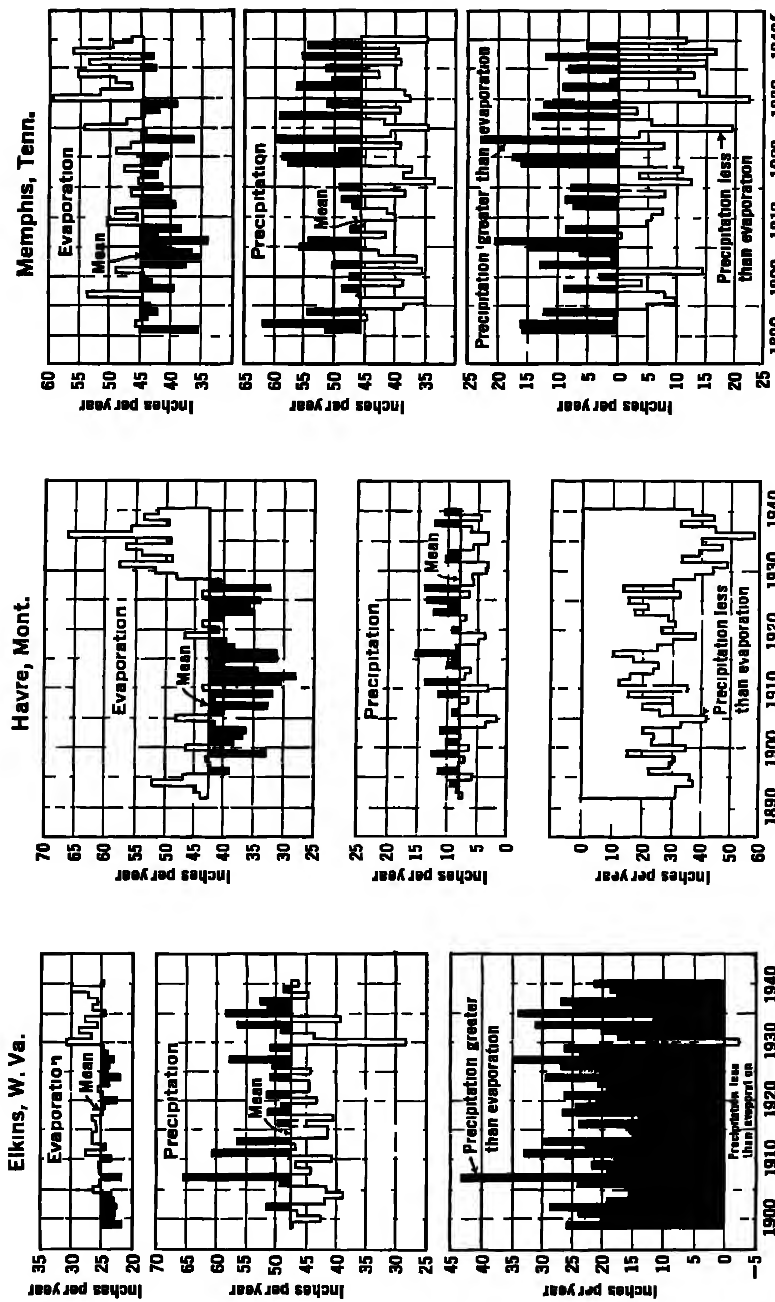

FIG 3 Difference between evaporation and precipitation in various parts of the country

3. Evaporation from Land Surfaces and Transpiration. Evaporation from land surfaces and transpiration are governed by the same climatological factors as evaporation from water surfaces; but evaporation decreases with the extent of vegetable shade, and transpiration increases with the hours of sunlight during the growing season. Both are greater on watersheds sloping to the south and subject to the direct force of prevailing winds.

In the West, where most of the stream flow, exclusive of floods, is derived from melting snow, where the plateaus, plains, and bottomlands contribute little water to streams, and where irrigation provides the major supply of moisture on cultivated lands, the term "consumptive use" has been applied to the evaporation and transpiration loss from land areas. This term includes not only the moisture transpired by the plants or retained in the plant tissue, but also the rainfall and irrigation water evaporated from the soil and the surfaces of the vegetation. For a detailed dissertation on consumptive use the reader may consult Refs. 8 and 9 of Bibliography, Section 9.

The character, condition, and extent of vegetation have a direct bearing on the rates of evaporation and transpiration; but the relation is not well established. The authors' careful examination of the published results of many experiments and of the opinions of numbers of writers has led to the conclusion that no general agreement has been reached. Mead * has said that "Observations in Wisconsin indicate that little change occurs in the flow of streams after deforestation . . . but that about the same amount of water is vaporized by the second growth and the crops or other vegetation on deforested areas."

It would seem, therefore, that at the present time one must assume the preponderating influence to be that of temperature and evaporation opportunity, the latter being affected by precipitation and the nature of the geological and topographical features of the watershed.

4. Effect of Character of Rainfall on Evaporation Opportunity. The percentage of evaporation from short, light showers is quite excessive, particularly if followed by sunshine. Heavy rains run off quickly, whereas light, continuous precipitation is conducive to infiltration and percolation. As temperature, more than any other factor, controls evaporation, winter precipitation produces the greatest percentage of runoff. The amount of evaporation increases with the annual rainfall, but the percentage of evaporation decreases with the rainfall.

Snowfall and frozen rain remain for long periods subject to evaporation, but they occur during seasons when the rate of evaporation is relatively small. This form of precipitation, therefore, is favorable to large runoff.

For individual rain storms, the evaporation increases with the amount of rainfall until the surface of the ground becomes thoroughly saturated. If rainfall increases above this point, the rate of evaporation remains practically constant; i.e., practically all the additional rainfall above this point is disposed of by infiltration or appears as runoff in the stream.

* See Ref. 10 of Bibliography, Section 9.

5. Rate of Surface Runoff. *Evaporation varies inversely with the rate of surface runoff, or directly with the length of time that surface water is subjected to the influences governing evaporation.* The average slope of the watershed is the principal factor governing the rate at which the surface water passes to the streams.

All plant growth affects, to some extent, the rate of surface flow. Forest cover is considered by many to reduce flood flows because it offers great obstruction to surface flow. In all probability, the dense underbrush accompanying forest growth has the greater influence. However, this effect may be reversed if forest shade delays the melting of snow until the heavy spring rains set in. The relatively greater effect of forests on delayed runoff is probably negligible during normal rains.

For each rainfall, practically a constant amount of water is intercepted by vegetation and later is evaporated without reaching the ground. The amount would naturally be greater for denser growths, but the proportion of the rainfall thus held back is small except for very light rains.

Lakes and swamps, because of their great retarding effect on the rate of surface runoff, afford the greatest opportunity for evaporation. In some climates the evaporation from such areas may greatly exceed the rainfall.

6. Facilities for Infiltration. Evaporation varies inversely with the porosity of the soil. Sandy soils greatly increase the infiltration and hence the low-season runoff, and tend, to a great extent, to equalize the flow of streams.* The depth of the pervious layer of soil is as important as the porosity of the soil; for, unless the water percolates to depths beyond reach of capillarity and absorption by roots, much will be returned to the surface and evaporated. Clayey soils and rocky slopes are not conducive to infiltration. Plowed ground is one of the greatest agents of infiltration, because, by the action of the plow, the hard top layer of the soil is loosened and its capacity for absorbing water is increased. Plowed fields should therefore increase the low-season stream flow by ground storage, provided the depth of soil is sufficient to conduct the water beyond reach of crops.

Flat slopes, lakes, swamps, and all vegetation retard flow and promote infiltration but, at the same time, provide greater evaporation opportunity unless the soil is quite pervious. Such agencies, therefore, tend to promote regularity of stream flow but reduce the total runoff.

Ground that is presaturated by long, heavy rains suffers a material reduction in capacity for infiltration. This is one of the principal contributing causes of excessive floods. Frozen and particularly ice-covered ground forms an efficient barrier to infiltration. As a result, many winter rains in northern latitudes appear entirely as stream flow and are, indeed, sometimes augmented by melting snow.

7. Recapture of Soil Moisture. A considerable portion of the rainfall, having disappeared as seepage, is returned to the surface by capillarity and through the roots of vegetation. Percolation is resisted by capillarity, which

* See Section 1, Chapter 3.

tends continually to supply the surface of the soil with moisture from the water-bearing strata. According to Hazen, the height to which water will be lifted by the capillarity of the soil varies inversely as the square of the size of the individual grains. Evaporation, through recapture, therefore, varies inversely with the size of the soil grains and the depth of the water-bearing soils.

Clayey soils not only resist infiltration and percolation by their relative imperviousness but also have greater capillary attraction. Hence, such soils are not productive of runoff by seepage or ground-water flow. Sandy soils, on the other hand, promote seepage, resist capillarity, and produce well-regulated high-percentage runoff.

The major portion of the water used in transpiration is derived through the root systems from the soil. Deep root systems have, therefore, the greatest capacity for recapture. For this reason heavy forest growth is considered to have a relatively unfavorable effect on the extent of permanent or deep seepage during the growing season. This is true only if the water table is relatively low, because, for high water tables, an equal opportunity for recapture by transpiration exists for the smaller root systems.

8. Irrigation Water. The amount of water used for irrigation and the percentage which returns to the stream depend upon the nature of the crops, the surface and subsoil, the amount of water available, and the irrigation methods in use. Unless detailed knowledge of the existing or proposed irrigation project is available, it is better to assume that at least two-thirds of the water used for irrigation is lost through evaporation and transpiration, although tests have indicated that, under certain conditions, 65 per cent may be returned to the stream [15].

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MAP OF UNITI

Red line

By courtesy U. S. Geological Survey

ES SHOWING MEAN ANNUAL RUN-OFF

s indicate average annual run-off in depth in inches

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CHAPTER 3

FACTORS AFFECTING RUNOFF

1 General A knowledge of the factors affecting the runoff characteristics of drainage areas is particularly essential when, in the absence of streamflow records the recorded runoff of a neighboring stream is used to estimate the probable amount and distribution of runoff of a stream, to be developed for power by a comparison of the relative runoff characteristics of the two watersheds. This chapter therefore treats of the practical application of this knowledge to such comparisons.

The disposition of rainfall the sole source of runoff, is described in Section 1 Chapter 1. The percentage of rainfall that appears as runoff at the site of proposed use varies widely. Figure 1 indicates the average annual runoff in the United States. The runoff from individual areas differs considerably from the averages shown depending upon local conditions.

For streams without complete artificial storage the distribution of the total runoff becomes of paramount interest as affecting the water available for power.

The extent of ground storage or the ability of the ground to absorb a part of the rainfall to be delivered to the stream weeks or months later in the form of seepage governs to the greatest extent the distribution of runoff. Most of the flow of streams during periods of drought is derived from ground storage.

Evaporation is by far the most important factor influencing the percentage of rainfall that finds its way to the stream. The conditions affecting evaporation are indicated in Fig 1, Chapter 2. The rate of evaporation is governed principally by temperature but the amount of evaporation and its seasonal distribution in which the engineer is most interested, are controlled principally by the geological formation of the watershed.

Water is sometimes diverted from the drainage area for municipal and industrial water supplies for other water power, for irrigation, or for other purposes. Such amounts can usually be closely determined and deducted from the computed runoff if they occur above the point of proposed use.

2 Deep Seepage Deep seepage is that portion of the runoff which passes through the underlying earth or rock strata below any possible intercepting cutoff at the dam, or which seeps through the divide into another watershed. It is usually inconsequential but cases have been cited where seepage from a very small drainage area into an adjacent drainage basin has caused serious losses.

For small drainage areas, a careful comparison of the geological formation with that in adjacent valleys should always be made, to determine the possibility of deep seepage.

Figure 2 is a section through two watersheds and shows how deep seepage may augment the flow of one stream to the detriment of another The percentage of rainfall lost by deep seepage into adjacent watersheds becomes rapidly less as the drainage area increases

In some sections of this country, the geological formations are such that large subterranean cavities exist in the bedrock and receive part and sometimes all of the flow of the stream This is a special form of deep seepage.

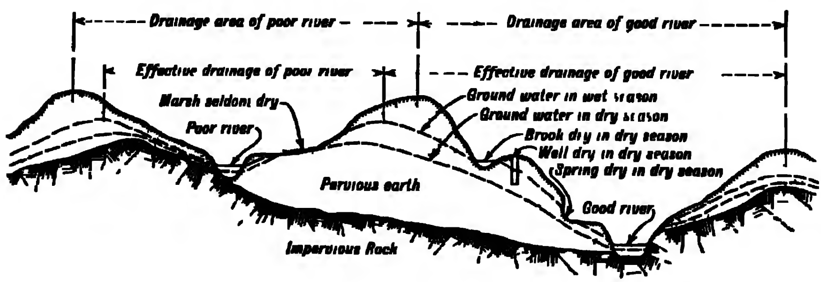

FIG 2 Wet and dry season ground-water stages showing deep seepage loss from a watershed

3. Geological Characteristics of the Watershed. Steep, impervious areas are productive of a large percentage of total runoff but are without adequate ground storage and hence are characteristic of small dry-season flows Hydrographs of such areas show flashy characteristics Rock surfaces are not necessarily impervious, although generally so Some classes of rock on relatively flat surfaces are quite pervious and offer excellent opportunity for ground storage Clayey surfaces and other shallow earth covering over impervious rock absorb little water.

The large cavities which exist in some limestone formations are excellent storage reservoirs of ground water if the earth covering is pervious Shales and shaley sandstone are quite impervious

Table 1, taken from the 1915 Report of the Water Supply Commission of Pennsylvania, indicates the extreme divergence of flow during times of drought, from adjacent drainage areas

Spring and Bald Eagle Creeks, in Center Co, Pa, have approximately equal drainage areas The remarkably better yield from Spring Creek is attributed entirely to the greater opportunities for ground storage afforded by the limestone formation which prevails generally throughout its drainage area. The better storage of Spring Creek is also indicated clearly by the uniform December and January flow, as compared with the increase from 8 to 68% runoff for the same months from Bald Eagle Creek following a wet period which embraced both areas

TABLE 1

COMPARISON OF LIMESTONE AND NON-LIMESTONE YIELDS DURING LOW PERIOD OF 1914–1915

Month	Spring Creek (145 Sq Mi), Limestone Formation		Bald Eagle Creek (140 Sq Mi), Non-limestone Formation	
	Discharge, sec-ft per sq mi	Per Cent of Runoff to Rainfall	Discharge, sec-ft per sq mi	Per Cent of Runoff to Rainfall
September	1.23	971	0.27	14
October	1.24	60	0.04	2
November	1.30	118	0.10	9
December	1.26	45	0.22	8
January	1.70	43	2.69	68

Flat, impervious areas are productive of irregular rates of runoff at all seasons. Stream flow from such areas is far from uniform but not so flashy as that from steep, impervious areas. Flat areas having coarse, sandy soil of considerable depth produce the most regular runoff.

The relative imperviousness of watersheds can be approximated by a careful inspection of the surface, in which consideration should be given to the extent of outcropping rock and of the subsurface conditions at ravines and excavations.

4. Extent of Lakes and Swamps. Increased discharge from a natural body of water is produced by a rise of water surface, which results in a greater depth and slope of the outlet channel This necessary rise of water surface provides temporary storage which retards the flow of the stream. The extent to which the flow will be equalized depends upon the extent of rise and the area of the water surface. Narrow outlets, requiring large fluctuations in stage for changes in stream discharge, are indicative of good natural storage and relatively steady runoff characteristics.

Although the tendency of all lakes and swamps is to equalize the flow, evaporation from the surface of water reduces the total runoff. The effect of evaporation is negligible during freshets and floods, and the maximum rate of runoff is materially reduced with the extent of lake and swamp area. During extremely dry periods, however, excessive evaporation may equal or exceed the surface flow and result in zero runoff if the area of water surface is a large percentage of the total tributary basin.

The percentage of runoff that is evaporated from water surfaces depends upon the proportion of area of water to land surface and upon the rate of

evaporation from the water surface. It was indicated in Section 2, Chapter 2, that evaporation from water surfaces decreases as the depth of water increases. Deep lakes are, therefore, productive of better regulation than shallow ponds and swamps. The extreme uniformity of flow of the Richelieu River, shown in Fig. 5, Chapter 10, is caused by the equalizing effect of Lake Champlain, which has considerable depth. Shallow lakes with wide outlets, and particularly swamp and marsh areas, though ordinarily beneficial to the regulation of ordinary and flood flows, are a menace to dry-season flows on account of extreme evaporation.

The equalizing effect of natural bodies of water during seasons of low flow depends principally, therefore, upon two conditions: first, the evaporation opportunity, which is fixed by the area of water surface in proportion to the tributary area and the depth of water; and second, the storage characteristics, which are dependent upon the absolute surface area and the width of outlet. The effect of these conditions may be beneficial to good regulation or decidedly the reverse, depending upon whether evaporation or storage has the greater influence.

The effect of lake and swamp area should be evaluated according to whether the rate of minimum flow or that of ordinary flow has the greater influence on the value of the development. The revenue from the output of developments, without auxiliary plant and serving a primary market, is fixed by the rate of minimum run off and is therefore affected adversely by great extents of shallow lakes and swamp areas. Developments with auxiliaries, or serving a secondary power market, or having limited storage, depend upon the rate of ordinary flows and may be benefited by shallow lakes and swamps. Developments operating in conjunction with large storage reservoirs need little or no assistance from natural storage, and hence the output is reduced by shallow lakes and swamps in proportion to their effect on the total runoff.

5. Vegetation. The influence of the extent and nature of vegetation on the amount and distribution of runoff is felt principally through its effect on evaporation. It seems to have been proved conclusively that cultivation of pervious soils overlying deep, impervious strata promotes underground storage and hence uniform flow. Meyer says that, if the level of saturation is about 20 ft below the surface in clay subsoils and about 10 ft in sandy subsoils, the ground water is safe from all vegetation except matured forest cover.

The relative effect of cultivation and of forests on low-season runoff is much the same. Tillage of the ground for cultivation opens and roughens the hard, smooth surface and promotes seepage. Roots of the forest trees open pathways for seepage, not only in the ground, but also in bedrock by getting into and opening seams, and the humus cover of virgin forests holds back the flow and increases the opportunity for seepage.

Transpiration is greater in cultivated than in forest areas during periods of high ground water, and less during other periods. This is because the transpiration demands are greater in cultivated areas for equal ground supply; but the roots of forest trees extend to a greater depth.

Deforestation of steep, rocky slopes has a great tendency to decrease low flows. Flat areas having thick, sandy soil are probably benefited by deforestation because of the removal of the transpiration demands of deep root systems, while, at the same time, assistance in holding back flow to promote seepage into the soil is less needed.

The demands of all forms of vegetation on ground water are excessive during drought, and, when an impervious understratum holds the water table close to the surface, reduction of ground storage by transpiration is excessive, particularly for cultivated areas. On the other hand, cultivated areas with deep ground-water systems are good low-season flow producers, and cultivation of such areas is particularly beneficial if the slopes are steep.

The effect of vegetation on low-water flow is relatively small in comparison with the influence of the geological and topographical characteristics of the watershed.

6. Geographical Features. The dry-season runoff per square mile is considered to increase with the area of the watershed; that is, the natural regulation for large areas is better than for small ones. Figure 3, plotted

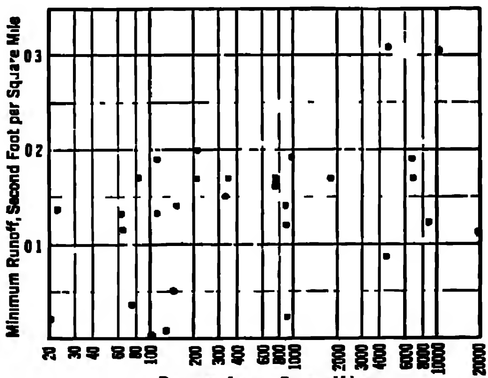

FIG. 3. Comparison of minimum flow with drainage area. North Atlantic Coast streams.

from data contained in the Geological Survey of New Jersey, 1894, indicates this tendency; but the relation is not well established for the territory covered by the data. For small, rocky watersheds and for small areas subject to deep seepage, the low-water flow may be negligible; but, for the streams plotted in Fig. 3, the variation in low runoff per square mile is probably caused by greater frequency of rainfall in the larger areas. While minimum flows per square mile increase with the area, flood flows per square mile decrease as the area increases. It would, therefore, seem that the effect of the size of drainage area on ordinary flows and total runoff is less than for minimum and maximum discharges.

Other things being equal, evaporation is less for higher elevations. Moreover, under certain conditions explained in Section 1, Chapter 1, rainfall increases with the elevation above sea level. Therefore, total runoff per square mile of watershed may be expected to be greater for the higher elevations. Hazen [1] has found that a rough approximation for inches of runoff from land surfaces in Massachusetts may be expressed by the following equation:

$$R = 19.4 + 0.0064E$$

where E is the average elevation, in feet, of the watershed above sea level. From this equation it would seem that the total inches of runoff at an elevation of 1000 ft above sea level would be 33% greater than at sea level. The relation applies to land areas only, and was deduced by correcting the observed total runoff on the assumption that 3 sq mi of water surface, on account of greater evaporation, produce as much runoff as 1 sq mi of land area.

Higher watersheds are usually steeper and less pervious. Consequently, though greater total runoff may usually be expected, dry-season runoff is likely to be much lower because there is less opportunity for ground storage.

7. Temperature. The effect of temperature on the amount and distribution of runoff is felt principally through its influence on evaporation, as previously explained. There are, however, several ways in which freezing temperatures can have an important bearing on the runoff.

In northern climates, the absence of the usual fall snows will allow the ground to become deeply frozen, and ground water, if close to the surface, may become solidified and unavailable for winter runoff. If the ground freezes below the plane of saturation, very little winter precipitation can be absorbed by the soil, the precipitation remaining on the surface in the form of ice and snow or passing off as surface flow. Between periods of rainfall, in such cases, the flow of streams is produced entirely from ground water which decreases in rate of flow until the spring thaw opens the surface to seepage flow. Frozen ground is not usually impervious, unless covered with ice or unless freezing has reached ground water. A heavy coating of early snow will protect the ground from deep freezing and, if the depth of freezing has not reached ground water, winter thaws will be absorbed by the soil.

When ice first forms on the surface of streams, it suddenly increases friction, and some of the flowing water is used to build up slope and area of waterway to maintain the required discharge capacity. The result is a sharp drop in discharge which continues until the winter regimen of the stream is established.

In extreme northern climates, lakes and streams of little slope freeze to a considerable thickness, and the amount of water thus held in storage by streams and shallow lakes forms a large part of the winter flow. W. G. Hoyt [2] cites an example showing the possibility of thus holding back the equivalent of more than 20% of the winter flow over a two-months' period following the freeze-up. Ice forming on deep lakes has no material effect

on stream flow if the capacity of the outlet is not thereby reduced. The freezing of very shallow streams in wide valleys may extend below the bed of the stream, in which event the ground flow is effectually cut off and the discharge reduced to surface runoff.

8. Seepage and Other Losses. Serious loss of water occurs frequently from dams on porous foundations, particularly in the arid regions, and from very long, unlined canals.

In the absence of rock formations, seepage from reservoirs depends upon the elevation of the water table and the nature of the underlying geological formation. The water table in the arid regions is at considerable depth, and as much as 30% of the flow has been lost where a dam has been constructed on particularly pervious foundations. Excessive seepage has sometimes required the lining of the bottoms of small reservoirs.

A. P. Davis [3] gives Table 2, showing seepage losses from the Deer Flat Reservoir. It will be noted that the seepage steadily decreased with the

TABLE 2

SEEPAGE LOSSES IN DEER FLAT RESERVOIR, IDAHO

Year	Mean Acreage Submerged	Maximum Acreage Submerged	Evaporation in Acre-feet	Total Loss in Acre-feet	Seepage Loss in Acre-feet	Average Seepage Loss	
						Acre-feet per day	Inches per day
1909	1,355	2,500	4,750	57,600	52,850	39.0	1.28
1910	3,002	3,900	10,500	95,483	84,983	28.3	0.93
1911	4,459	6,300	15,600	150,838	135,238	30.3	1.00
1912	4,625	7,000	16,200	85,089	68,889	14.9	0.49
1913	5,250	8,200	18,400	89,489	71,089	13.5	0.44
1914	5,337	8,400	18,700	82,084	63,384	12.0	0.40
1915	5,123	8,100	17,900	67,400	49,500	9.7	0.32
1916	4,820	6,900	16,900	43,970	27,070	5.6	0.18
1917	4,500	7,550	11,000	32,400	21,400	4.8	0.16
Mean	4,274	6,540	14,440	78,150	63,820		
Total	38,471	58,850	129,950	704,353	574,403		

length of service. The improvement is attributed to the filling of the subsoil with water, resulting in a rise of the water table, and not to any actual tightening of the soil.

Certain rock formations make poor foundations for high dams. Davis states that gypsum deposits and limestone deposits showing evidences of caves are productive of excessive leakage, and that volcanic rock and coarse-grained sandstone should be regarded with suspicion.

In arid regions the losses from unlined canals and reservoirs may be excessive when the underlying materials are pervious. From a study of experiments by E. A. Moritz [4] and others, Etcheverry, in Table 3 [5], gives the

TABLE 3

CONVEYANCE LOSSES IN CUBIC FEET PER SQUARE FOOT OF WETTED PERIMETER FOR CANALS NOT AFFECTED BY THE RISE OF GROUND WATER

Character of Material	Cubic Feet per Square Foot in 24 Hours
Impervious clay loam	0.25 to 0.35
Medium clay loam underlaid with hardpan at depth of not over 2 to 3 ft below bed	0.35 to 0.50
Ordinary clay loam, silt soil, or lava ash loam	0.50 to 0.75
Gravelly clay loam or sandy clay loam, cemented gravel, sand, and clay	0.75 to 1.00
Sandy loam	1.00 to 1.50
Loose sandy soils	1.50 to 1.75
Gravelly sandy soils	2.00 to 2.50
Porous gravelly soils	2.50 to 3.00
Very gravelly soils	3.00 to 6.00

average conveyance losses* in unlined canals of ordinary ratios of width to depth, the larger values being for canals less than five years old.

Most of the experiments upon which Table 3 is based were made in canals in the arid regions, where the ground water is very low and is present during the irrigation season only. Canals for water power operate continuously, and the wet-season seepage losses would ordinarily be considerably less, owing to high ground water.

The foregoing applies only to conditions existing in the arid regions of the western United States where the ground-water level is frequently at a very great depth below ground surface. Even there, any particular case may depart widely from the figures given in Table 3. If the position of ground water, the topography of the surface, and the effective size and porosity of the material are determined, it is entirely possible to compute, for any particular case, with a fair degree of accuracy, the amount of seepage which may be expected. The methods to be used for such determinations of seepage are described in detail in "The Design of Earth Dams," by Joel D. Justin, *Trans. A.S.C.E.*, Vol. 87 (1924).

* Including about 5 to 10% of evaporation losses.

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CHAPTER 4

RIVER GAGING

By N. C. Grover and the late John C. Hoyt†*

1. Introduction. The following statement, covering the essential principles involved in systematically measuring and recording the flow of water in surface streams, will serve the needs of the engineer who uses records of stream flow in designing and operating hydraulic power plants. Not only the measuring of discharge and the recording of stage are involved but also the establishing, maintaining, and operating of the stations where the records are collected and the converting of the field observation data into systematic records. For more complete details of many phases of the collection of systematic records of river discharge, reference may be made to *River Discharge* [1], by Hoyt and Grover; *Stream Flow* [6], by Grover and Harrington; and "Stream-gaging Procedure" [7], by Corbett.

The basic field data required for the computation of daily flow of a stream are daily records of stage and measurements of discharge covering the range of stage, from which a rating (stage-discharge) curve may be developed to show the discharge corresponding to any stage.

Computation of daily discharge of a surface stream is based on the axiom that the discharge at any given stage will be unchanged as long as the flow is steady and the bed and banks of the stretch of river controlling the stage-discharge relation at the gage remain unchanged, and that the discharge will vary with the stage according to some natural but fixed law. This stable relation between stage and discharge makes practicable the construction of a rating (stage-discharge) curve (Fig. 10) from a few measurements of discharge that are well distributed over the range of stage. This rating curve is the graphic representation of the formula for computing the discharge past the gage. Any change in the conditions affecting the stage-discharge relation at the gage will make it necessary to construct a new rating curve. If there is no change in these conditions, measurements made in different years will plot on the same curve, which may be applied to a record of stage extending over the whole period to obtain estimates of daily discharge.

The stage-discharge relation will be stable when there exists below the gage an effective "control" section in which the bed and banks are perma-

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nent. The control may be found: (1) at the head of rapids where the fall is large; (2) at a succession of two or more riffles at each of which there is a moderate fall; (3) in a long stretch of the channel of a stream having a low gradient; or (4) at an artificial structure in the channel. Natural stability of the stage-discharge relation can be obtained only by careful selection of a site where the bed and banks of the river channel below the gage are so stable that the stage-discharge relation does not change because of erosion or deposition of silt or drift. Obviously, a site should not be selected above a dam or other structure containing openings through which the flow is controlled by gates that may be open or closed, so that the stage is not a true index of discharge.

The sensitiveness of the stage-discharge relation is important. A station should be selected so as to give as large a change in stage as is possible for a given change in discharge. Estimates of flow at stations that have relatively small fluctuations in stage are liable to large errors on account of lack of refinement in the determination of mean daily stage The rate of fluctuation of stage at the gage should be such that the least change in discharge to be recorded, say 1% of the total flow, should cause a readable change of stage. The sensitiveness will be determined by the shape of the control. A broad, flat control will cause a small fluctuation in stage for a large change in discharge, and a notch-shaped control will cause a large fluctuation in stage for a relatively small change in discharge.

The stage-discharge curve for an ordinary gaging station does not differ greatly in appearance from the discharge curve for a weir. Although parabolic in form, it may not be a simple parabola like that for a weir, as the section or sections that determine the relation of stage to discharge are irregular in shape. As indicated above, however, the stage-discharge relation for any river channel with permanent control section will be as fixed as that for the flow over a weir. The curve may be accurately defined by a few current-meter measurements of discharge, and, when it is so defined, records of stage may be converted into records of discharge without appreciable error.

2. Establishment of Gaging Stations. The selection of a satisfactory site for a gaging station involves finding (1) a site for the gage where the stage-discharge relation will be stable and (2) a section or sections for making the discharge measurements in which the current is smooth and the velocity is measurable—that is, between 0.5 and 15 ft per second—at all stages of the river. If time permits, all possible sites for the gage should be inspected at high as well as at low stages and in both winter and summer, before final choice is made and the gage installed.

The selection of a site where a bridge is available for supporting the engineer while he is making observations of depth and velocity may be desirable as a matter of economy in equipping the station, but few bridges occupy the best sites for making measurements of discharge. Moreover, bridge piers disturb the smoothness of the current and therefore decrease the accuracy of

the determinations of velocity. Hence some degree of accuracy is generally sacrificed when a bridge is chosen as a site for making gagings. The erection of a cable and car outfit for use of the engineer in measuring discharge involves considerable expense, but it may be more than justified because it yields more accurate records.

The site for the gage itself is most important. Whether it is of the recording or nonrecording type, the gage must be accessible at all stages and must be so situated that it will not be unduly exposed to danger of damage from drift. If possible the gage should be so placed with respect to normal currents that drift or silt will not be deposited in such a manner as to cut off its open connection with the free surface of the flowing water. The most critical section of a river related to a gaging station is therefore that section for which the stage-discharge relation is to be determined or the section in which the gage is placed, and not necessarily that in which measurements of discharge are made. Such measurements are frequently made in some section other than that of the gage, and not uncommonly different sections are used for measuring the flow at different stages of the river, in order to obtain the highest degree of accuracy. Where the flow is steady—that is, neither rising nor falling—the same quantity of water is passing each section of the river, and measurements of discharge made in other sections apply equally well to the section of the gage. The essential requirement is stability of the section or sections controlling the stage-discharge relation at the gage.

Shifts in bed and banks, either in the section of the meter measurements or in the gage section, will not affect accuracy of records, provided that such shifts do not extend to the control section, causing changes in the stage-discharge relation at the gage. This statement may be illustrated by comparison with a weir. The relation of head on the weir to the quantity of flow over the weir is controlled by the shape and height of the crest and is not affected by changes in the bed or banks of the pool above the weir if those changes are not sufficiently great to affect materially the magnitude or distribution of the velocities of the approaching water. Accurate current-meter measurements of discharge made within the pool would plot on the discharge curve of the weir even if considerable changes in the bed or banks or both occurred between successive discharge measurements. The pool above a dam or weir is not, however, a good site for a gage, even if the dam and its foundations are tight and no water is discharged through or around the dam, because at such a site the relation of stage to discharge will not be sensitive at low stages.

In order that any method involving the conversion of observations of stage into records of discharge may be satisfactorily applied, the percentage of flow that causes a readable increment or decrement of stage must not exceed the percentage error allowable. For example, if a record is desired that is accurate within 1%, a single second-foot of water must make a readable difference in stage when the flow is 100 sec-ft. This degree of sensitiveness cannot ordinarily be obtained in a pool above a dam with a level crest.

3. Equipment of Station.* The site having been selected, the establishment of the gaging station consists of the installation of a gage for observing or recording stage and a cable or other structure from which measurements of discharge may be made; the placing of bench marks for referencing the gage datum; and, at many sites, the erection of an artificial control for maintaining the stage-discharge relation (Fig. 1).

The gage must be so placed that it may be read easily and accurately at all stages of the river and that its datum will remain constant. It must be

Fig. 1. Typical cable gaging station and caging house with automatic register.

referred to bench marks entirely removed from the gage or the structure on which it is placed, in order that the gage, if disturbed, may be replaced to the same datum. The gage should be graduated and should generally be read to hundredths of a foot at low stages of the river and to tenths or half-tenths of a foot at high stages.

The type of gage used will depend on many factors. A staff gage, either vertical or inclined, is perhaps the most common. Such a gage may be painted directly on wood, or preferably, if vertical, it may be stenciled and enameled on sheet metal which should be attached to a plank or timber.† It must be rigidly attached to solid rock, a masonry bridge pier or abutment, or the foundation wall of a building at the water's edge. It should not be attached to a tree at the water's edge because of danger that the datum of the gage may be changed if the roots of the tree are undermined or that the

* Plans and specifications for equipment for gaging stations are available at the U. S. Geological Survey.

† Enameled gages may be obtained from the Baltimore Enamel Co., Baltimore, Md.

gage may be lost if the tree is carried away at high water. If attached to wooden piling the datum must be carefully watched, especially if the river has a heavy ice cover in winter. If no suitable rock or structure is available, the gage may be attached to masonry piers built for the purpose. Such piers should have their foundations below the frost line and be otherwise so constructed as to be in minimum danger of heaving. An inclined staff gage must be graduated by use of a Y level after it is placed in final position, and, as no two inclined gages will have exactly the same slope and therefore the same length of graduation, the use of enameled-metal facing for such a gage is not practicable. An inclined gage may be made up of two or more sections of different slope.

Fig. 2. Wire-weight gage.

Staff gages are difficult to maintain in rivers subject to heavy ice cover, to large quantities of drift, or to log driving. In such rivers wire-weight gages are frequently used because they are entirely removed from danger of damage by ice, logs, or other drift.

The wire-weight gage (Fig. 2) consists of a wire with weight attached, which is lowered from a fixed position into contact with the water surface. The distance from the fixed position to the water surface is measured by the amount of wire let out, as indicated either by a counter on a drum or by a graduated horizontal staff.

The installation of a water-stage recorder of the float type (Fig. 7) requires a table that is rigidly set for supporting the register, that will not shift in elevation, and that is above the highest stage of the river; a float well so connected with the flowing water of the river that the stage of water in the well, which is, of course, the stage recorded, will at all times be the same as the stage of water in the river; a shelter to protect the gage from rain, dust, and inquisitive or malicious molestation; and, in regions of frost, such protection for the well, or in some localities such provision for heating the well, as to remove the danger of formation of ice around the float. The gage shelter and well must be so situated as to be free, if possible, from danger of destruction by ice, logs, or other drift; and so constructed as to withstand the strains of such pressure and knocks as cannot be avoided.

If these fundamental requirements are followed, many entirely satisfactory styles of wells and shelters may be built, plans for which may be obtained on application to the U. S. Geological Survey, Washington, D. C. A durable type is the concrete well and shelter (Fig. 1), which is in many respects most

satisfactory, having outside dimensions of 6 ft by 6 ft and affording ample space for operating the recording gage and entering the well. It should be equipped with a ladder leading from the trap-door to the bottom and solidly attached to one wall or built as an integral part of it. Both corrugated or

FIG. 3. Measurement by wading.

other iron pipe and cement pipe have been satisfactorily used for a well, especially in regions of no frost Such pipe wells should be large enough to permit a man to enter from the top if side entrances cannot be arranged.

The intake pipe should be 3 to 5 in in diameter, set normal to the current and always fully open for the passage of water, either inward with a rising stage or outward with a falling stage In clear-water streams there is no difficulty in keeping an intake pipe open In silt-laden streams special precautions may be necessary If the topography permits, the intake pipe may be omitted and a direct connection made between the well and the river.

In order to keep the intake pipe open a flushing device is used consisting of a riser pipe and valves so arranged that it can be filled with water by either a pump or bucket and then released to clear the intake pipe.

Generally, float wells on silt-laden streams become filled more or less rapidly with a deposit of silt, which must be removed either through the trap-door in the well-house floor or through a door or doors in the side wall of the well. The silt must be removed so promptly and effectively that the graph record of stage made by the recorder will be accurate and complete.

FIG. 4. Measurement from bridge.

The cost of installing a recorder will vary widely according to the range of stage of river to be provided for, the topography of the bank in which the well is to be constructed, the nature of the material to be excavated, the remoteness from sources of supply of materials and labor, and the protection that must be provided against the formation of ice in the well and damage to the well and shelter by ice or other drift. A shallow well, for a range of stage of a few feet, and a small wooden shelter may be built for $200 or even less under favorable conditions. In remote localities installations built of concrete, with wells excavated largely in rock and providing for a 50-ft range in stage, have cost as much as $20,000 each. The cost of ordinary installations, however, ranges between $300 and $1500 in addition to the cost of the water stage recorder.

In northern latitudes the well must be set some distance into the bank, and the exposed part, if built of boards, must be carefully battened to prevent the formation of ice around the float. If electric current is available

the installation in the well and the operation in winter of a small electric heater, or even the constant burning during cold weather of one or two electric lamps, will generally eliminate frost. If electric current is not available, a false floor in the well just below the frost line will generally prevent freezing. Such a floor must be provided with openings through which the float and weight cables can be passed, and must be built so that it will float on the surface of the water if the stage rises above the supports.

At each gaging station equipped with a water-stage recorder there should also be a nonrecording gage, generally a staff, either vertical or inclined, which should be read and recorded whenever the station is visited. Such a reading will disclose whether the stage of the water in the well is the same as the stage of the river. If it is not, the cause of the discrepancy, generally clogging of the intake pipe, should be removed, and a full record made of exactly what has been done and the effect produced on the record of stage, which is later to be converted into a record of discharge.

The datum levels of all gages at a station should be coincident and should be referred to permanent bench marks so situated as to be accessible at all stages of the river and convenient for use in checking the gages by means of a Y level.

Measurements of discharge may be made from a bridge (Figs. 4 and 9), cable and car, boat, or by wading (Fig. 3). When available at suitable measuring sections, bridges are most commonly used. Boats have no general use because of the expense and difficulty involved in handling them. If cable and car are used, measurements can usually be made more readily than from a bridge or a boat and it is possible to place the station at the best measuring site. Cable installations of spans up to 2000 ft are in common use. A-frames of wood or steel are used to support the ends of the cables up to heights of 40 ft. Above 40 ft, four-legged supporting towers are needed. Special designs should generally be made for the large installations. Anchorages for holding the cable should be made of concrete, and all connections with them should be above ground so that they can be easily inspected. Standard plans for cable installations and for other equipment needed at river measurement stations may be obtained from the U. S. Geological Survey, Washington, D. C.

4. Measurements of Discharge. The methods ordinarily used for determining the quantity of water flowing in open channels are of two kinds—the velocity-area method and the weir method. In the former the quantity of discharge is obtained as a summation of the products of partial areas of the cross-section of the flowing water by the respective measured velocities in such areas; in the latter the discharge is computed by a formula (or obtained from tables computed by a formula) from the observed "head" on the weir and the known dimensions of the weir. The velocity-area method is now almost universally used in river gaging because of its comparative cheapness and its essential reliability and applicability under a wide range of conditions, because of the excessive cost of constructing and rating weirs or dams solely

for river gaging, and because of the inaccuracies involved in using any except standard sharp-crested weirs with free overfall.

5. Weir Method.* The weir method involves the use of a formula which contains three factors—area of cross-section, velocity, and a coefficient varying with the type of weir. This method is therefore a velocity-area method, but it differs essentially from other velocity-area methods in that the weir is generally rated—that is, the relation between stage and discharge is established—by comparison with the laboratory rating of a model. Its accuracy depends primarily on the faithful reproduction in the field of the conditions that prevailed in the laboratory where the rating was made. A weir rated in place is seldom used for river gaging.

The essential purpose served by a weir used in river gaging is to maintain a stable relation between the discharge and the stage of water above the weir. This relation has been determined for standard sharp-crested weirs, with and without end contractions, by means of many experiments. The Francis weir formula without end contractions, the most commonly used in this country, is

$$Q = CBH^{3/2}$$

in which Q = the quantity of discharge in cubic feet per second;
C = the coefficient equal to 3.33 for sharp-crested weirs, but varying for crests of other shapes;
B = the length in feet of the horizontal crest of the weir;
H = the depth in feet of water flowing over the weir [2].

For convenience in use, tables of discharge for sharp-crested weirs of various lengths have been prepared and are available in many handbooks. Many investigations [3 and 4] have been made to determine coefficients for use in the Francis formula for the discharge over dams having various shapes of crest. The results have shown a wide range in the value of coefficients, even for dams having crests of similar shape. Estimates of flow at dams are made even more inaccurate by leakage and by errors in measuring or estimating the water diverted for use in power plants or through waste and flood gates, log sluices, and fish ladders. The collection of accurate records of flow by the weir method is therefore limited to the use of sharp-crested weirs, which are feasible to construct only where small quantities of water are to be measured and enough of the head to operate the weir can be sacrificed. Such conditions are not commonly found at or near power plants but are characteristic of irrigation canals and distributaries.

The weir method depends for accuracy on the construction of a weir without leakage, with a crest of permanent shape and elevation, of sufficient length and with abutments of sufficient height to carry the largest quantities of water to be measured without exceeding the limit of depth of flow over the crest for which the weir coefficient has been determined. The weir must be short enough for the smallest amount of water to be measured, say 1% of the flow,

* See also Sections 11 and 12, Chapter 8.

to cause a readable change in stage. Uncertainties are large in assuming coefficients for use with the many forms of dams to be found in rivers. Aside from all other sources of error, there is a possibility of an error of 10 to 20% in selecting the coefficient to be used, and the result is a corresponding percentage error in the computed discharge. Rating curves for dams with crests that are long in relation to the quantity of water flowing over them will lack in sensitiveness and will yield inaccurate results on that account. Moreover, the cost of computing records collected at power dams, complicated by partial discharges through wheels of different sizes and operated with varying gate openings and through waste, flood, and other gates, is frequently much greater than the cost of establishing and operating velocity-area stations rated by current meter. On account of these conditions the use of dams for river gaging is not recommended.

6. Velocity-area Method. The velocity-area method involves the measurement of cross-sectional areas and velocities of the flowing water. The cross-sectional area of the stream is determined by measurements of width and of depth at points spaced to show the shape of the bed and the summation of the partial areas computed from these measurements. Sufficient lead must be used to obtain correct soundings. Velocity may be measured by slope, float, or current meter, each involving a fundamentally distinct process.

7. Slope Measurements. The determination of velocity by a measurement of slope has been described in Section 10, Chapter 8.

In measuring discharge by the slope method it is necessary to determine the mean area of cross-section and the slope of the surface of the stream and to observe the roughness of the bed and banks, which will determine the proper value of the discharge coefficient. In making such a measurement a straight channel, 200 to 1000 ft long, must be selected and measured. In this stretch the slope and cross-section should be reasonably uniform and the bed and banks should preferably be permanent. The slope must be sufficiently large to be measured without a large percentage of error.

Errors in applying the slope method, which may aggregate 25% or more, are of three classes: (*a*) errors in the value of slope used in the formula, due either to incorrect observations of the fall in the water surface, which is generally so small that minor errors in leveling or in estimating the position of the water surface make large percentage errors in the result, or to changes in slope in the stretch of river used; (*b*) errors due to the selection of wrong values of the coefficient to be used in the slope formula; and (*c*) errors in the area of cross-section, which should be the true average of all cross-sections in the stretch.

Slope measurements of velocity have two practical uses: (*a*) in estimating flood discharge, generally after the crest of the flood has passed and when the data available are the slope and area of cross-section as determined from marks along the banks and a knowledge of general conditions of channel and banks during the flood; and (*b*) in designing canals for which the slope must

be determined in order that the channel may carry a certain quantity of water at a given velocity.

8. Float Measurements. The float method of measuring velocity of water flowing in natural channels is direct, is more accurate than the slope method, and is less accurate than the current-meter method. It may be used without special equipment and in rivers carrying so much drift that a current meter cannot be used. Surface floats are generally used because some substance that may serve as a float is almost always at hand, either on the water itself during floods or on the banks at other times. Generally, where subsurface or tube floats can be obtained, the current meter can also be made available, and the increased accuracy to be had by using other than surface floats can be attained better and at less cost by means of the more accurate current meter. Tube floats measure the mean velocity directly, with a coefficient of nearly unity, and are used satisfactorily in artificial channels of uniform cross-section. They are not adapted to use in natural channels. Velocities obtained by using surface floats must be reduced to mean velocities by coefficients varying between 0.75 for low stages in shallow channels and 0.95 for high stages in deep channels, and averaging about 0.90.

Errors in applying the float method, which may aggregate 15% or more, may result from: *(a)* failure to obtain a true average area of cross-section of the stretch of river used; *(b)* failure to obtain properly distributed observations of velocity; and *(c)* choosing a wrong coefficient to reduce observed velocities to mean velocities.

9. Current-meter Measurements. The current-meter method is very common for measuring the velocity of water flowing in an open channel. It combines reasonable cost and practically universal applicability with reasonable accuracy, and it may be used on streams of all sizes and depths with velocities ranging from 0.5 to 15 ft per second. The measurements may be made from a boat, bridge, or cable, or, if the depths and velocities are not too great, by wading. The velocity of the flowing water is measured at several points in a single cross-section. Each measured velocity is multiplied by the appropriate partial area to obtain a partial discharge. All partial discharges are combined to give the total discharge. Errors may be involved in the rating of the current meter and in the observations of its action, but as all the measurements are made in one cross-section this method avoids the error, common in slope and float measurements, resulting from the averaging of velocities over a stretch in which the area of cross-section is varying. As the current meter under proper conditions of use will measure velocity within a probable error of 2%, the measurements are more accurate than those generally obtained by other methods.

For measurements with a current meter, a section of the river must be chosen in which the water flows smoothly and the velocity is reasonably uniform throughout the cross-section. Measurements should not be attempted in turbulent water, as meters of all types are affected, either accelerated or retarded, by vertical motion of the water. The failure of a current meter to

operate accurately in such water is as well established as the failure of the Y level to operate accurately in unsteady air. A cross-section should be sought where the current is reasonably regular over the whole width. Such a section can generally be utilized by means of a cable to support the engineer at medium and high stages and by wading at low stages.

The principal sources of error in current-meter measurements of discharge, which should never exceed 5% and in good work are held below 2%, are in the rating of the meter, in observations of soundings, in placing the meter in position, in observations of revolutions of the meter, in observations of time, and in the use of insufficient observations of velocity.

Ability to place a meter accurately in position depends largely on the use of sufficient lead to reach the bottom without deflection and of the smallest possible cable, for supporting the meter, that is consistent with proper strength.

Meter ratings do not change rapidly so long as the meter is not damaged by accident, and material errors in rating between velocities of 1 ft and 15 ft per second are rare. In general, especially in measuring low velocities, error will be reduced to the minimum by observing the time (not less than 45 seconds) of some number of complete revolutions of the meter. A stop watch recording to fifths of a second should be used for observing time.

A current-meter measurement of discharge is an essential part of river gaging and is made to determine the position of one point on the rating curve for the gaging station. In order to serve this purpose, both the stage and the discharge must be measured accurately. The gage must therefore be accurately set with respect to its datum and accurately read, at least at the beginning and end of the current-meter work and once each hour during the progress of the work.

The distance from an initial point on one bank of the stream to the points at which depth is measured to determine area of cross-section must be measured and marked on the bridge or cable from which the measurement of discharge is made, in order that the position of the meter with respect to the initial point may at all times be definitely known and recorded and that the cross-section of the flowing water may be accurately computed and the appropriate velocity applied to each partial cross-section. If a wading or boat measurement is made, a measured and tagged line must generally be stretched across the stream to serve the same purpose.

Sufficient observations of depth and velocity must be made to disclose true area of cross-section and true velocity of water in each partial area. In general, observations will be made at points 1 or 2 ft apart in streams up to 30 or 40 ft wide, 5 ft apart in streams up to 150 ft wide, 10 ft apart in streams up to 400 ft wide, and 20 to 50 ft apart in wider rivers.

There are several methods of making discharge measurements, varying with the number and position of the points at which velocity is observed. In the vertical velocity-curve method, observations of velocity are made at a sufficient number of depths, not less than five, at each point, to define the

curve of velocities (Fig. 5) at that point. The mean velocity is obtained by averaging the velocities scaled from the curve or by dividing the area between the curve and its axis, as determined by planimeter, by the depth. This method is used as a standard and may be made as accurate as desired by increasing the number of points of measurement. It requires too much time, however, for ordinary use, especially on streams that are of considerable size or are likely to fluctuate in stage and discharge during the measurement.

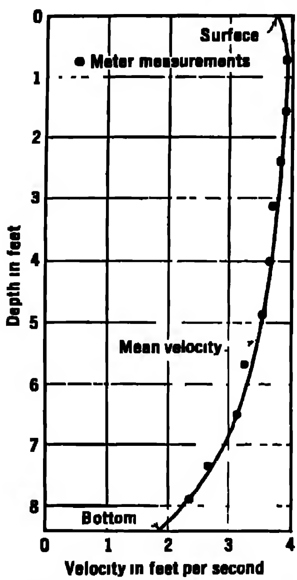

Fig. 5 Typical vertical velocity curve

Vertical velocity curves show that the distribution of velocities in the vertical varies as the ordinates to a parabola, and that the velocity at 0.6 depth and the average of the velocities at 0.2 depth and 0.8 depth are very nearly the average of all velocities in the vertical. For this reason it is possible to measure approximately the average velocity in any vertical by one observation at 0.6 or, more accurately, by two observations at 0.2 depth and 0.8 depth.

After the measurement is completed, the pivot bearings of the meter should be wiped dry and oiled before the meter is returned to its box. As the pivots are made of hardened steel they will rust if not properly cared for, and the rating of the meter will be changed.

If the stage of water is unusual or if the gaging station is not readily and cheaply accessible, the meter measurement should be computed before the station is left. If the plotting of the discharge indicates either change of rating of the station or error in the measurement, the current-meter work should be repeated with great care in order to establish beyond doubt a point on the rating curve.

10. Current Meters and Accessory Equipment. Current meters for measuring the velocity of water in rivers are of two types, the direct-action and the differential meter. Many meters of both kinds have been built, most of which have been designed for use under special conditions. The only ones that have had any great general use are the Ott * and Haskell † direct-action meters, and the small Price differential meter.‡ The small Price meter (Fig. 6) has had the most general use and is regarded as the universal meter because it alone satisfies the requirements of general regional river gaging. When equipped with a penta-head, which is a device for indicating every fifth revolution, the meter may be used for measuring velocities ranging

* Manufactured in Germany.
† Manufactured in Ithaca, N. Y.
‡ Manufactured by W. & L. E. Gurley, Troy, N. Y.

from 0.5 to 15 ft or more per second. It may be operated from a boat, bridge, or cable, or by wading, for measuring the discharge of rivers of depths ranging from 0.5 to 50 ft or more. It may be operated by one man, who also keeps the notes, and may be carried with all accessories by one person traveling on foot or horseback. It is not easily damaged and with reasonable

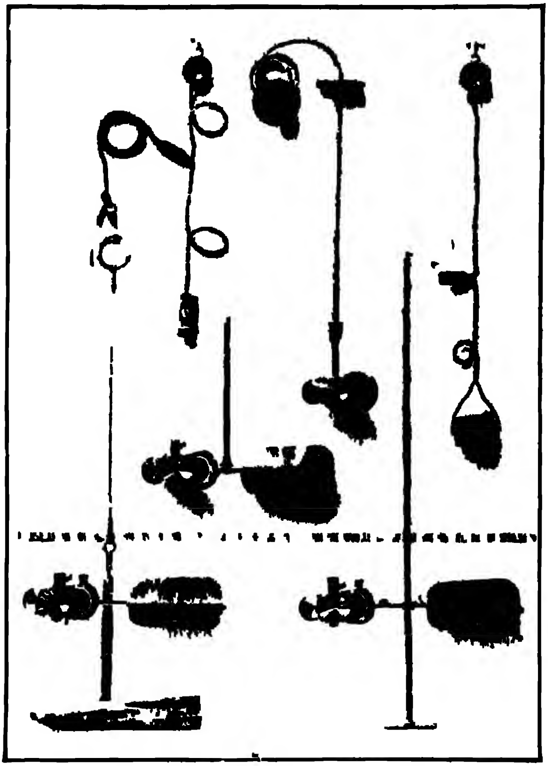

Fig. 6. Price current meters.

care does not change in rating. There may be other meters that are somewhat more accurate for measuring turbulent water, but no other meter has yet been developed for universal use. Effort is constantly being made to produce a meter that will measure accurately velocities less than 0.5 ft per second and that may be used reliably in turbulent water. Such a meter must be adapted to the requirements of general use, as indicated above, before it can supplant the small Price meter for this work.

The current meter must be equipped with a device for indicating revolutions of the meter wheel. A sounder is preferred to a recorder, because it makes any failure to act or any irregularity in action of the meter immediately apparent to the operator, who can then correct the difficulty and

thus avoid error in the measurement of discharge. A meter that is sufficiently sensitive for measuring low velocities satisfactorily must be designed to indicate only every fifth revolution, in order that the revolution may be countable when the meter is used in currents of high velocity. In general, an electric buzzer operated by a small dry-cell battery is used.

In order to promote accuracy of sounding and to aid in keeping the current meter in proper position, specially designed torpedo-shaped weights ranging from 15 to 300 pounds are used. Either hand-operated or power-operated reels and booms (Figs. 8 and 9) are used with the larger weights.

11. Rating of Current Meter. Current meters are rated by drawing them through still water at known or observed velocities and counting the revolutions. Although meters of the same kind have essentially the same rating, individual meters vary from the average by small percentages. Each meter must therefore be tested and its rating determined. This is done by the National Bureau of Standards, Washington, D. C., for a small charge, and manufacturers will deliver their meters through the Bureau of Standards in order that the purchaser may obtain a true rating at minimum cost in time and money.

12. Records of Stage. Records of stage are collected by one of the various types of gages indicated in Section 3, Equipment of Station. The

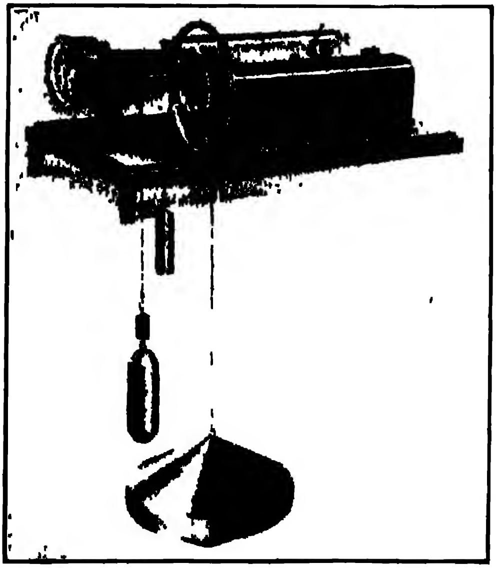

FIG. 7 A typical water-stage recorder.

simplest form of gage is the graduated staff, from which direct readings of the height of water are obtained. In order to insure accurate records, water-stage recorders are essential, and they are now used at most river-measure-

ment stations. Pressure gages, operated by transmission of pressure of the various depths of water through a diaphragm to an air chamber connected with the recording device by a flexible tube, have not given satisfactory service because of temperature and other disturbing changes. Water-stage recorders like the one shown in Fig. 7, commonly used in gaging rivers, record the stage in the form of a graph, with time on one rectangular axis and stage on the other. Several good graph registers are on the market. All are of the float type. The graph with rectangular coordinates has an advantage over a polar graph in that the time scale is not reduced at low stages, when records are generally most valuable. The principal recorder manufacturers are W. & L. E. Gurley, Troy, N. Y.; Julien P. Fries & Sons, Baltimore, Md.; and Leupold, Voelpel & Co., Portland, Ore.

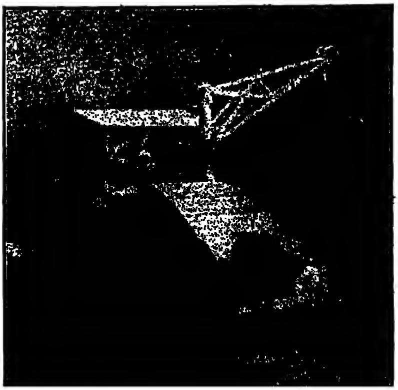

FIG. 8. Movable crane for handling Price meter with heavy weight.

13. Operation of Gaging Station. The operation of a river-measurement station consists of the collection of a record of stage and measurements of discharge, together with such activities as may be needed to insure the reliability of these two essential factors of the record. If a nonrecording gage is used, a reliable gage reader must be employed to read and record the stage once or twice daily. If a 7-day automatic recorder is used, someone must be employed to wind the clock and change the record sheet periodically. If a weight-driven clock and a 60-day or continuous recorder are used, it may be cheapest and most satisfactory to have no one touch the instrument between visits of the engineer. Even under these conditions, however, a local man may well be employed to visit the station at stated intervals to see that the clock is running and to report on the conditions of ice or drift in the vicinity of the gage.

Sufficient measurements of discharge must be made to define accurately the stage-discharge curve. For this purpose each measurement must be made as carefully and accurately as possible. Inaccurate measurements throw doubt on the exact position of the curve and serve no useful purpose. Measurements should, if possible, be made at a constant stage. If made at a changing stage the result will be a sum of partial discharges for the different stages that have occurred during the measurement but will correspond to no single stage and may have little value in defining the true position of the stage-discharge curve. Measurements should be well distributed over the range of stage, with the greatest number in that portion of the curve where the curvature is changing most rapidly. The upper and lower ends of the station-rating curve are most likely to be poorly defined, on account of the

brevity of periods of extreme high and extreme low water, especially the former, and the difficulty in reaching the gaging station during such periods. The lowest point in the control section or, if that cannot be easily found, the lowest point in the cross-section at or near the gage will probably correspond approximately with the stage of no flow, that is, the stage at which the stage-discharge curve will intersect the gage-height axis. This point is extremely valuable in helping to define the position and shape of the lower part of the rating curve.

The stage corresponding to each measurement of discharge should be obtained from the gage from which is made the record of stage that is to be used in computing the record of discharge, as that gage must be used in determining the stage-discharge curve for the station.

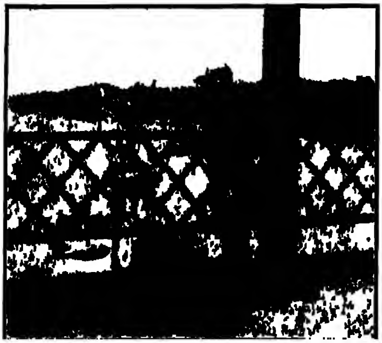

FIG. 9 Light hand-operated crane for handling Price meter.

The station having been well rated, occasional careful measurements of discharge are needed to insure the continuous applicability of the rating. Even the most carefully selected and accurately rated stations will sometimes change sufficiently, perhaps temporarily, to disturb the stage-discharge relation. Besides shifting beds and banks, there are local and temporary obstructions, like the lodging of ice or other drift in shoal places or against a projecting rock, that may cut off part of the channel and so bring about a higher stage for a given discharge. In some rivers, growing water plants give much trouble. As the summer advances the plants become larger and cause progressively greater obstruction to the flow and progressively higher stages of river for the same quantity of discharge. With the approach of winter the vegetation disappears, and the stage-discharge relation becomes the same as in previous winters. Such temporary changes in rating will, if undiscovered, lead to errors of considerable magnitude in the record of discharge.

The gage or gages must be maintained in position. The datum of each gage must be checked at least once a year by Y-level comparison with a bench mark near by. If the gage is inclined, several points on it must be compared with the datum, because of the danger of changes in inclination caused by heaving or settlement of one or more of the supporting posts or piers. If a chain gage is used, the length of the chain must be checked and corrected at least once a year as an essential part of the maintenance of its datum. The graduations of the gage must be kept clearly legible by washing the face of the gage if it becomes dirty, or by repainting or replacing the face of the gage if the markings become defaced or so faint that they are not easily seen.

14. Winter Records. During periods of partial or complete ice cover, the regular stage-discharge relation is disturbed in varying amounts depending on the extent and thickness of the ice. Such disturbance has the nature of back water and is always manifested by an increase in stage for a given discharge. For those periods it is generally better to correct the records of stage by subtracting the amount of the back-water effect rather than to attempt to prepare a new rating curve for each change in conditions affecting the stage-discharge relation. The amount of the correction for back water must be obtained by occasional current-meter measurements of discharge and by study of the progressive increases in stage with the increases in extent and thickness of the ice, including comparison of changes in stage at gaging stations affected by ice with changes at stations in the same region that are not so affected.

Current-meter measurements of discharge under ice should be made either by the vertical velocity-curve method or by measurements at 0.2 and 0.8 depth. The vertical velocity curve under ice will have a form somewhat different from that of the open channel but will still be parabolic, and therefore the average velocity will be correctly measured by averaging the velocities found at 0.2 and 0.8 of the depth from the bottom of the ice to the bed of the stream.

15. Computation of Daily Discharge. The conversion of the measurements of discharge and the record of stage, which together constitute the

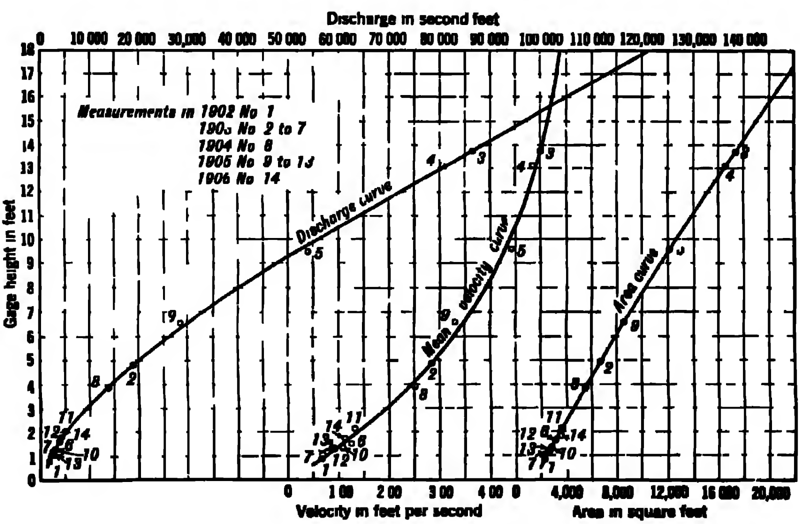

FIG. 10. Discharge, mean velocity, and area curves, Potomac River at Point of Rocks, Md.

essential field data, into a record of discharge involves the construction of a station-rating curve (Fig. 10) showing the stage-discharge relation applicable to the gage used in obtaining the record of stage, the conversion of this curve

into a rating table for convenient and accurate use, the application of the record of stage to the rating table to obtain a record of daily discharge, and the computation, preferably by ordinary computing machines or Crelle's Tables, of records of monthly and annual runoff. Most of these steps involve only care in computation. In the construction of the stage-discharge curve, however, both judgment and care are necessary in adjusting the curve among the plotted discharge measurements, which may be more or less discordant. Judgment must also be exercised in choosing the methods of determining the mean daily stage to be used in computing the mean daily discharge or, if the fluctuations are too great and rapid for using an average stage for the day, in deciding on the interval of time, hourly or longer, that may be used without undue sacrifice of accuracy in the computed results.

An instrument has been devised for the mechanical conversion of a water-stage-recorder graph into a record of discharge. The instrument, with the rating curve of the station set accurately into it, is operated like a planimeter. The pointer is made to follow the gage-height graph, and the discharges are read from the planimeter wheels.

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CHAPTER 5

ESTIMATING STREAM FLOW

1. General. All estimates of stream flow are based upon stream-gaging records, obtained at the site, near the site, or on neighboring streams. It is not possible to estimate stream flow solely upon rainfall records with a degree of accuracy sufficient for practical purposes.

The main purposes for which estimates of stream flow are made may be stated as follows:

1. To estimate the average annual energy output of the development.
2. To estimate the additional energy and the increase in firm capacity provided by a storage reservoir.
3. To estimate the minimum annual energy output and firm capacity.
4. To determine the capacity of a storage reservoir for equalizing the flow to a given minimum.
5. To estimate the minimum daily output without storage.
6. To estimate the necessary capacity of a flood control reservoir for limiting discharge to a predetermined maximum under all conditions except extreme floods.
7. To estimate the necessary spillway capacity.

In Table 1, Chapter 1, is given the probable deviation of short-term precipitation records from the true mean. There is reason to believe that stream-flow records are subject to no greater errors. This applies to the probable errors that are involved when runoff records are used to determine average annual energy output and the average annual energy provided by storage reservoirs, items 1 and 2, above.

In the case of estimates of flow governing minimum annual output and minimum daily output, and estimates intended to determine the capacity of reservoirs to equalize the flow to a given minimum, the problem is one of probabilities, the general solution of which is given in Section 4, Chapter 6. The minimum yearly and daily output can be defined only in terms of probabilities, the occurrence of which may be expected not oftener than once in a given period of years. The determined size of the reservoir, to equalize the flow to a given minimum, is that size which will be deficient not oftener than once in a given period of years.

The methods of computing the amount and distribution of stream flow which may be used to develop power vary with the extent and nature of available data. Such data may be grouped as follows:

1. Long-term stream gagings at the site.
2. Short-term stream gagings at the site.
3. No stream gagings at the site.

All stream-flow records should be corrected for possible diversion from the watershed for irrigation, water supply, and other purposes, and for increased evaporation losses if large storage reservoirs are to be used.

2. Stream Gagings at the Site.* Although stream-gaging stations have been established at thousands of places in the United States, records of stream flow at single stations covering a long term of years are comparatively scarce.

The most important data on stream flow are published in the *Water Supply Papers* of the U. S. Geological Survey, Washington, D. C. The Corps of Engineers, U. S. Army, and many of the states also publish yearly tabulations of stream gagings and general studies of various streams. A few reliable records may also be obtained from water-power and water-supply companies and city water-supply commissions and engineers.

The U. S. Weather Bureau publishes daily river stages of many streams, without data as to discharge. Studies of a number of large streams may be found in the Tenth Census of the United States, Vols. 16 and 17.

The results of stream-flow measurements are given in the *U. S. Geological Survey Water Supply Papers*, which are published annually in 14 parts, each part covering an area whose boundaries coincide with natural drainage features as indicated below:

PART
1. North Atlantic slope basins.
2. South Atlantic slope and eastern Gulf of Mexico basins.
3. Ohio River basin.
4. St. Lawrence River basin.
5. Upper Mississippi River and Hudson Bay basins.
6. Missouri River basin.
7. Lower Mississippi River basin.
8. Western Gulf of Mexico basins.
9. Colorado River basin.
10. The Great Basin.
11. Pacific slope basins in California
12. Pacific slope basins in Washington and upper Columbia River basin.
13. Snake River basin.
14. Pacific slope basins in Oregon and lower Columbia River basin.

Other *Water Supply Papers*, published periodically, contain data on water resources of special districts, including studies of hydrology, hydrography, flood flows, analyses of river waters, water powers and water-power possibilities, underground flow, river profiles, and other data of interest. A list of all *Water Supply Papers* may be obtained by writing to the Director, U. S. Geological Survey, Washington, D. C. A list of rivers that have been

* See Chapter 4 for description of stream-flow gagings.

gaged in each district, and of the years covered by the individual records, may be found at the end of each of the later *Water Supply Papers* applicable to the district in question.

The probable accuracy of stream gagings is usually stated in the reports. A factor of safety should always be applied to results obtained from such records, to cover all possible inaccuracies of measurements and, as explained previously, to compensate for the possibility that the years of record do not correspond to average conditions or include years of reasonably low flows.

3. Extensions of Stream Gagings at the Site. The period of stream gagings may be extended to include additional years, by establishing a relation between the flow indicated by such gagings and the simultaneous flow of a neighboring stream on which records cover a longer period. In Fig. 1 is given a runoff-relation curve for the Little Swatara and Brandywine Creeks, Pa., for three years of simultaneous records. Each dot indicates a concurrent average monthly discharge per square mile of drainage area. After the whole period of overlapping records has been plotted, an average curve is drawn through the points. This curve is used to alter the longer records of the neighboring stream to indicate the probable flows in the stream under consideration for years for which records are not available. The better low-season runoff of the Brandywine and the better wet-season runoff of the Little Swatara are clearly defined in Fig. 1.

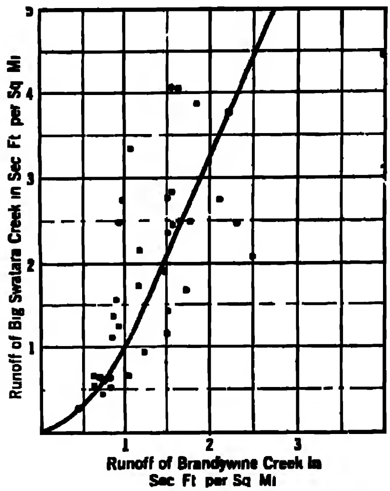

FIG. 1. A typical runoff-relation curve.

Suppose that on a stream, A, under consideration, there are 10 years of records and on a neighboring stream, B, there are 20 years of records, 4 years of which overlap those of stream A. If a runoff relation is established between the two streams, as previously described, and this relation is used to extend the records of stream A, the total period of estimated stream flow for A would be $10 + 20 - 4 \quad 26$ years.

The probable error involved in the use of only the 10 years of record of stream A, if the estimate is used for average output, is indicated by Table 1, Chapter 1, 5-state group, to be about 5.6%. If a perfect runoff relation were established between the two streams, the resulting 26 years of record would have a probable error of only about 2.7%.

The following equation gives the maximum permissible error in the estimated runoff relation between the two streams to result in a benefit from the extension of the records of stream A.

$$x_3 = \frac{100y_2}{y_2 - y_1}\left(\quad \frac{100 - x_1}{100 - x_2}\right)$$

where x_3 = the permissible error in the runoff-relation curve;
x_1 = the probable per cent error in the records of stream A if used alone;
x_2 = the probable per cent error in the extended records of stream A if a perfect runoff-relation curve were used;
y_1 = the years of record of stream A;
y_2 = the years of record of stream B.

Substituting in this equation, we have

$$x_3 = \frac{100 \times 20}{20 - 10}\left(1 - \frac{100 - 5.6}{100 - 2.7}\right) = 6.00\%$$

If it is thought that, in establishing the runoff relation, there may be an accumulative error of 6% or greater, no benefit will be derived from the extension and the 10 years of stream A should be used alone. The probable error in the runoff-relation curve can be based only on the judgment of the engineer and is affected considerably by the length of simultaneous gagings and the relative runoff characteristics of the two streams. The smaller the average distance of the dots from the curve, the greater the degree of accuracy. The curve of Fig. 1 is fairly well defined but a probable error of 10 or 15% would not seem unreasonable. The curve of Fig. 2 is poorly defined and might be in error as much as 20 or 30%.

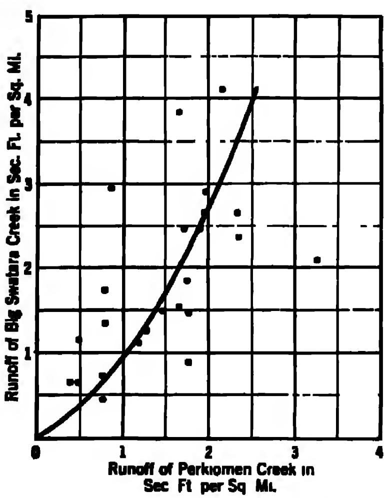

FIG. 2. A poorly defined runoff-relation curve.

If the runoff characteristics of the two streams are materially different, a more exact runoff-relation curve may be established by plotting a runoff-relation curve for each season or for each month. In such cases, weekly averages may be used instead of monthly averages, in order to obtain a greater number of points from which to estimate the curve.

A number of methods have been proposed* for estimating the amount and distribution of stream flow from an established relation between rainfall and runoff and applying this relation to long-term rainfall records. Such methods cannot be of any value unless the relation between rainfall and runoff is based on at least several years of simultaneous runoff and rainfall records. Most of the methods are rational but are not intended to be exact. Because of the

* See Bibliography, Section 6.

numerous factors affecting the relation between rainfall and runoff, which have not yet been coordinated to an extent sufficient for exact analysis, it is probable that the errors involved in extending runoff data by this means may be very great, and the chief objection to the use of such methods is the difficulty in determining the probable degree of accuracy. Moreover, those methods that include a consideration of all major agencies affecting runoff involve an amount of labor and time which ordinarily cannot be devoted to such problems.

In Table 1 is given a relation between the rainfall and runoff of the Miami watershed above Dayton, Ohio.* Rainfall and runoff years were used in

TABLE 1

ANNUAL RAINFALL, RUNOFF, AND TEMPERATURE, MIAMI RIVER ABOVE DAYTON, OHIO

(1) Year Ending Sept. 30	(2) Inches Rainfall	(3) Degrees Fahrenheit Temperature	(4) Actual Inches Runoff	(5) Inches Runoff Computed from Equation of Section 4
1894	30.7	54.7	4.9	10
1895	34.0	53.0	3.7	12
1896	46 2	54.0	8.1	16
1897	33.3	53.4	12.8	8
1898	44 3	55.0	14.7	15
1899	34.2	53.2	9.7	9
1900	35.1	54.4	6.6	9
1901	30.1	53.4	5.6	7
1902	31.6	51.1	5.8	10
1903	37.1	53.6	12.6	11
1904	39 1	49.8	13.1	15
1905	38.5	51.5	7.1	13
1906	33.2	52.9	9.2	9
1907	43 1	51 9	17.2	16
1908	37.7	53.1	17.7	11
1909	39.3	53.2	13.1	12
1910	36.3	52.3	15.1	12
1911	39.8	53.7	13.9	12
1912	43.8	50.8	23.1	18
1913	42.9	54.0	24.4	14
1914	32.3	53.3	8.3	8
1915	41.8	52.1	12.1	15
1916	39.9	53.2	19.2	13
1917	35.7	51.1	11.4	12
1918	36.8	50.3	9.4	14
Mean	38.07	52.76	11.87	12

* See Ref. 21, Section 9, Chapter 3.

establishing the relation. Columns 2 and 4 are reproduced in Fig. 3 of this chapter.

Figure 3 is typical and indicates that the variation in the amount of annual runoff is affected more by the distribution of the rainfall during the year than by the amount of rainfall. The average annual temperature, indicated in Col. 3, was so nearly constant that it probably had little influence in the variation of runoff. It is evident that such a relation, if used for extending the period of runoff records, may lead to considerable error for individual years although the general average may approximate the truth.

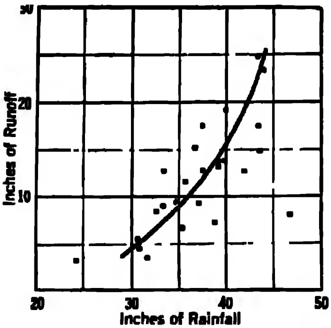

FIG. 3. Relation between annual rainfall and runoff, Miami River, above Dayton, Ohio.

4. No Stream Gagings at the Site. If no stream gagings have been made at the site of the proposed development, it is necessary to base estimates of stream flow on other gagings in the vicinity. If gagings have been made on the stream a short distance from the site, the problem presents no features of great difficulty. In such cases it is considered that, if the two drainage areas have no appreciable differences in rainfall and runoff characteristics, the runoff per square mile will be the same for each.

The area of the watershed may be obtained from various sources. The topographical maps of the U. S. Geological Survey usually supply the best information obtainable A number of other maps have been published by the Survey. The U. S. Post Route maps and various other state and county maps are available for this purpose. Surveys are sometimes necessary if accurate maps are not available.

If the only gagings available are those made at a considerable distance from the site, and particularly if they are on another stream, estimates of stream flow at the site must be based on such gagings, modified to compensate for differences in the areas and the rainfall and runoff characteristics of the two watersheds. A practical solution of this problem calls for the best judgment of the most experienced engineers, backed by a careful examination of the drainage areas, a study of all published data relating to their physical and climatological features, and an intimate knowledge of the effect of the many controlling factors mentioned in the foregoing sections.

The errors that may result from the assumption of the same runoff per square mile from two drainage areas, without modifications for differences in the characteristics of the two watersheds, are well indicated by the comparison between the flow per square mile from the Big Swatara Creek and Brandywine Creek watersheds, shown in Fig. 1. Errors as great as 100% may be expected in some cases. Figure 1 clearly indicates the relative

imperviousness of Big Swatara Creek, resulting in greater total runoff but less opportunity for ground storage, and hence small low-season flows.

If gagings have been made on the stream both above and below the site of the proposed development, estimates of stream flow at the site can often be made with considerable accuracy by interpolation.

Short-term gagings on the stream relatively near the site should always be used in preference to longer-term gagings on a different stream, because, for the former, parts of the watersheds are identical. The short-term gagings on the same stream may be extended, however, by means of a runoff-relation curve as explained in Section 3.

If there are no gagings at the site or in the vicinity, reliable estimates of stream flow cannot be made. Though admitting the possibility of extending runoff records with some degree of accuracy by the establishment of a relation between rainfall and runoff by methods referred to in Section 3, engineers generally agree that it is impossible to establish such a relation without the aid of at least several years of runoff records, particularly for cases where it is necessary to know the distribution of runoff.

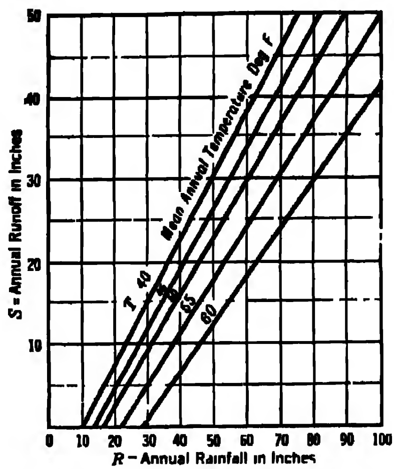

FIG. 4. Vermeule's approximate relation between annual rainfall, temperature, and runoff.

Vermeule's equation* for annual runoff is

$$S = R - (11 + 0.29R)M$$

where S = total annual runoff in inches;
R = total annual rainfall in inches;
M = a factor which depends upon the mean annual temperature.

Vermeule's recommended value of M has been used in plotting Fig. 4. This equation is the result of studies to determine the effect of forests on evaporation, and, though extreme accuracy is not claimed for it, the equation has received wide publicity.

Figure 5, showing a comparison between observed and estimated values of mean annual evaporation ($R - S$), computed from Vermeule's equation, has been plotted from his tabulations of rivers in this country and abroad. An error of 50% ordinarily would be possible in the application of the equation to determine the average annual runoff for a considerable period of years. In the arid region, greater errors might be made.

* See Ref. 9, Section 9, Chapter 3.

The runoff of the Miami River above Dayton, Ohio, computed by Vermeule's relation, is given in Col. 5 of Table 1. The observed and computed runoff, shown by Cols. 4 and 5, have been compared in Fig. 6. It is seen that Vermeule's equation may be several hundred per cent in error if used for single years, although a close agreement may exist between the observed and computed mean for the period.

In 1914 Justin [3] developed an annual runoff formula which takes account of average slope, mean annual rainfall, and temperature Fahrenheit. The formula has also been used to obtain monthly runoff by building up a synthetic mass curve. A group of watersheds having long-term, accurate records of rainfall, runoff, and temperature, scattered over a large area, was selected to supply the basic data upon which the formula is developed. The formula is applicable to the eastern United States and, in general, should give results within 10% of the true runoff. In more remote locations, with widely varying conditions, the formula would require revision to adapt it to the several variables. When applied with proper care and judgment to streams within the recommended area, this formula has closely agreed with recorded runoff on watersheds entirely outside the list used in the build-up of the formula. The formula is simple to apply and is arranged as follows:

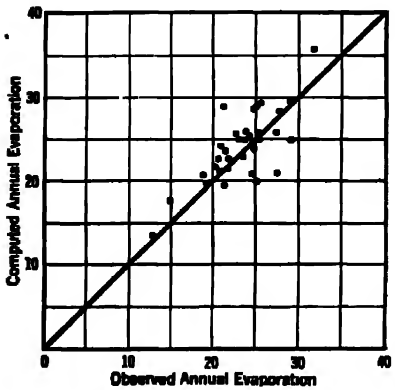

FIG. 5. Comparison of observed evaporation and evaporation computed by Vermeule's equation.

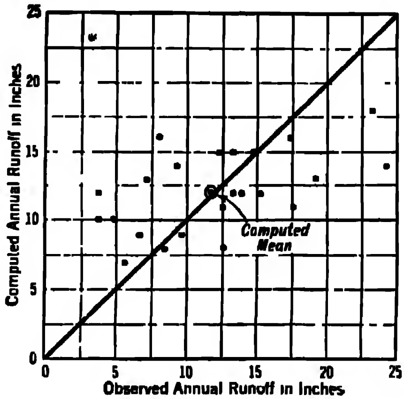

FIG. 6. Relation between observed runoff of the Miami River above Dayton Ohio, and runoff computed from Vermeule's equation.

$$C = 0.934S^{0.155}\frac{R^2}{T}$$

where C = the annual runoff in inches;
S = the average slope of watershed;
R = the mean annual rainfall in inches;
T = the mean annual temperature Fahrenheit.

5. Storage and Pondage. The demands for power ordinarily correspond in no degree to the varying natural flow of the stream. Therefore, unless a large part of the natural flow is to be wasted, storage and pondage must be provided to regulate the flow so that it may be made available at times when the generation of power is required.

Storage consists in the impounding of a considerable amount of the excess runoff during seasons of surplus flow for use during dry seasons. Ordinarily, the contents of storage reservoirs are fed out in such a manner that, when they are added to the natural runoff from the intermediate area between the reservoir and the site of the development, the resulting flow at the development will correspond as closely as possible to the flow demanded for power.

When the reservoir is some distance from the development, it is impossible to regulate the outflow with sufficient accuracy to provide for sudden changes in load demand or to compensate for the varying runoff from the intermediate area. Therefore, a regulating body of water, or pondage, is needed directly at the plant. This may be drawn upon quickly to suit sudden changes in load demand and to compensate for inaccuracies in operation of the storage reservoir. When storage reservoirs are not provided, pondage is necessary to regulate the natural flow to suit the variation in daily or weekly load demand.

Usually, the hourly demand is quite variable, and the average demand during a work day is often materially different from that of Saturdays and Sundays. The average weekly demand, however, is generally fairly constant. The duty required of pondage without storage is therefore usually that of regulating the weekly flow to suit the variation in load demand from the average weekly demand.

In general the word "storage" is used to indicate the building up of the low natural flow of the stream to a uniform discharge, and the word "pondage" is used to indicate the regulation of the resulting uniform flow, or the natural flow if there is no storage, to suit variations in weekly load demand. If storage is at the site of the development, it will also provide the necessary pondage.

6. Bibliography. (See also Bibliographies for Chapters 1 and 3.)

1. W. G. Hoyt and H. C. Troxell, "Forests and Stream Flow," *Trans. A.S.C.E.*, Vol. 99, p. 1, 1934.

2. H. W. Dennis, "A Method of Adapting the Records of Stream Flow at One Point to Another Point on the Same Stream," *Trans. A.S.C.E.*, Vol. 84, p. 551, 1921.

3. Joel D. Justin, "Derivation of Run-Off from Rainfall Data," *Trans. A.S.C.E.*, Vol. 77, p. 346, 1914.

4. Merrill Bernard, "An Approach to Determinate Stream Flow," *Trans. A.S.C.E.*, Vol. 100, p. 347, 1935.

CHAPTER 6

FLOOD FLOWS

1. General. It is necessary to know the characteristics of floods to be expected in order to insure the safety of the dam, the powerhouse, and other parts of the development and to acquire reservoir property for flooding rights.

The spillway of a high earth dam, above a populated district, must be designed to pass a flood of extremely remote probability in order to avoid the possibility of great loss of life and property damage. The spillway of a concrete dam on good foundations, similarly situated, may be designed for a somewhat smaller flood because, while an overtopped earth dam will surely fail, a concrete dam on good foundation has a material factor of safety.

The protection of the powerhouse and other appurtenant structures as well as the purchase of reservoir flooding rights can be based on a much smaller flood, for the reason that it is more economical to stand the damage once in a long period of years than to incur the initial expense of protection against a very large flood. Thus the determination of the proper flood to provide for involves the question of probabilities and judgment.

A method will be given to determine the probable frequency of the smaller floods, but the probable frequency of the large floods for use in the design of the spillway is not at present possible of determination and another method must be resorted to.

Where there is no storage at the site to smooth out the peak of the flood, the maximum rate of discharge is of primary interest, and the total volume in the flood and the shape of the hydrograph are seldom, if ever, of importance. However, if there is a large reservoir above the dam and if a rise of water surface above the spillway crest is necessary to pass the flood, a part of the flood will temporarily be held back by the storage above the spillway crest and the peak of the flood will be materially reduced. In this case the volume in the flood and the shape of the hydrograph are as important as the peak discharge and frequently more important.

The modern method of estimating flood runoff by the study of meteorological influences, infiltration losses, the unit hydrograph, and physical characteristics of the watershed involves a degree of research that is justifiable only for the final designs of important dams.*

* For a complete analysis see Ref. 2.

An approximate method sufficiently precise for preliminary studies will be included here.

2. Peak Flows. Table 1 will be found useful for a study of record-breaking floods in any section of the country. It contains a list of the peak flow of unusual flood discharges in the United States and other countries; it is an abridged form of a table compiled by Jarvis and Creager [2] in 1941. The table includes only those floods that are necessary to define an enveloping curve for each state. It excludes also a number of large floods when a larger flood has been recorded close by on the same river.

Floods from Table 1 having values of C, in Eqs. 4 and 4a, greater than 30 are plotted in Fig. 1. These data are in terms of the maximum momentary flow. The original records described a few floods in terms of 24-hr average flow and these have been adjusted for momentary flow by Fuller's equation [1]:

$$Q = Q_1(1 + 2A^{-0.3}) \qquad [1]$$

where Q_1 = the recorded 24-hr average flood in cubic feet per second;
Q = the corresponding momentary peak flow;
A = the drainage area in square miles.

This equation is admittedly approximate, being based on very few observations.

When the magnitude of the peak of a flood from a given drainage area is known, it is sometimes desired to know what the peak probably would be from a different drainage area of the same characteristics and under the same meteorological conditions—for instance, from another point on the same river or a similar river.

It is known that, with other conditions the same, the greater the drainage area, the smaller the flood per square mile of area, the most commonly used equation being

$$Q = C'A^n \qquad [2]$$

or its equivalent,

$$q = C'A^{n-1} \qquad [2a]$$

where Q = the flood in second-feet;
q = the flood in second-feet per square mile;
A = the drainage area in square miles;
n = an exponent which is less than unity;
C' = a coefficient depending upon the characteristic of the drainage basin.

A reference to Fig. 1 indicates that, with the exception of a few floods of rather remarkable magnitude, an enveloping curve can be drawn which indicates an approximate variation of flood peak per square mile with drainage

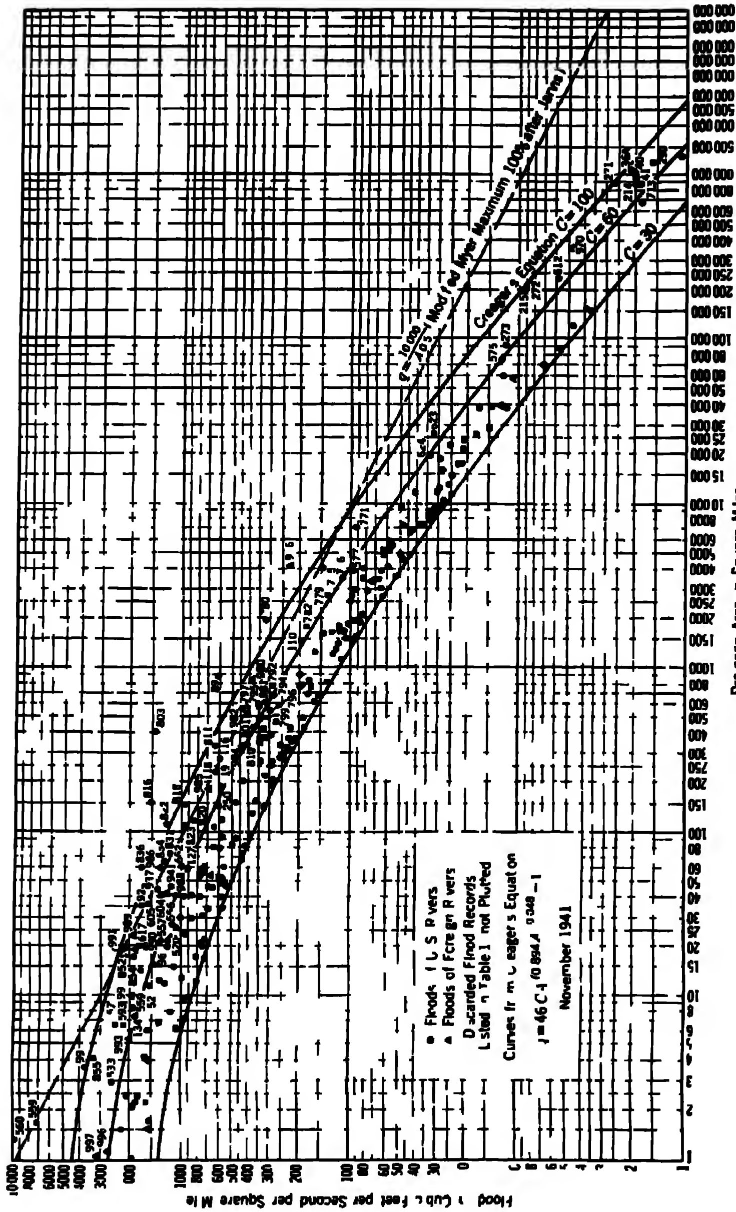

FIG. 1. Record of unusual flood discharges

area. Such a curve has been drawn in Fig. 1. It indicates that the value of n is not constant but takes the approximate form of

$$n = 0.894A^{-0.048} \quad [3]$$

and the equation of the enveloping curve * the form of

$$Q = 46CA^{(0.894A^{-0.048})} \quad [4]$$

or its equivalent,

$$q = 46CA^{[(0.894A^{-0.048})-1]} \quad [4a]$$

In Fig. 1 the enveloping curve for $C = 100$ follows the general trend of all maximum floods with the exception of those resulting from two storms: the 1940 storm in North Carolina (floods 559, 560, and 760 †) and the May-June 1935 Texas storm (floods 780, 789, 803, and 816). There was also an unofficial record of an extraordinary flood in the Philippines (976).

These floods are inexplicable. They were estimated by the U. S. Geological Survey, the Texas records being reviewed at the request of the authors. Therefore, it must be assumed that these floods were the result of meteorological conditions so different from those for usual great storms as to put the floods in a class by themselves, and hence they would not affect the general trend of the enveloping curve.

The "modified Myer's maximum 100%," which is the most used of the equations of the form of Eqs. 2 and 2*a*, has been shown in Fig. 1 solely for comparison with Eqs. 4 and 4*a*.

The equation of the enveloping curve for districts having smaller flood peaks than that corresponding to $C = 100$ has not yet been found, but it is believed that for the present Eq. 4*a* with C less than 100 is the best available. Equation 4*a* is useful in drawing enveloping curves for plotted floods in limited districts. Such curves can be drawn easily by multiplying the ordinates of $C = 100$ in Fig. 1 by a constant to obtain a curve for any desired value of C.

The first step in the use of Table 1 in flood studies comprises an investigation of the climatological features of the surrounding section of the country in order to determine the limits within which such features may be considered approximately the same as those for the drainage area in question. A search is then made for record floods throughout that section of the country, and these are plotted as in Fig. 1. An enveloping curve, plotted from Eq. 4*a* or reducing the curve for $C = 100$ in Fig. 1 as explained, would then indicate the flood characteristics of the maximum flood-producing streams in that section. One must be sure that the section considered is large enough, has had its share of floods, and has been sufficiently gaged for its records to be indicative. Otherwise a larger section must be considered, even though it extends into greater flood-producing sections. As the ultimate limit, Fig. 1 could be used.

* Equations 4 and 4*a* are not "flood formulas" in the usual sense.

† 11,200 sec-ft per sq mi from 0.4 sq mi is off the figure.

TABLE 1

Unusual Flood Peaks, United States and Foreign Rivers

By C. S. Jarvis and William P. Creager

( ) Items in parentheses are known to be approximate or unofficial.
Item numbers are the same as in the original table. (See p. 101 of Ref. 2.)

Item No	Drainage Area, sq mi	Flood, sec-ft per sq mi
	Alabama	
1	30,800	14.4
2	26,300	10.8
3	15,400	9.5
4	8,390	17.9
5	4,830	44.6
6	3,840	18.2
7	2,500	23.6
8	2,100	29.4
9	1,900	30.0
10	1,700	30.4
11	504	31.0
12	272	43
13	7.4	69
14	3.9	53
	Arizona	
19	225,000	1.05
20	56,000	3.93
21	17,750	7.5
22	12,000	24.7
23	6,260	22.0
24	6,000	27.6
25	5,756	36.0
26	3,850	23.4
27	1,480	66.3
28	544	82
29	450	110
30	340	104
31	210	95.4
32	200	125
33	30	440
34	20	647
	Arkansas	
40	1,050,000	2.31
41	1,000,000	2.04
42	150,300	3.7
43	51,500	(4.0) 6.35
44	19,000	16.8
45	1,540	91
	California	
51	22,500	25.0
52	9,300	31.8
53	3,627	51.6
54	3,070	95
55	1,940	56.3
56	1,921	95
57	1,201	99.9
58	1,060	75
59	845	118
60	655	91.6
61	619	162
62	613	101
63	608	88.3
64	565	169
65	510	133
66	465	275
67	394	176
68	375	187
69	325	231
70	299	241
71	262	336
72	254	220
73	249	223
74	227	187
75	222	248
76	219	174
77	209	280
78	204	440
79	189	201
80	172	250
81	140	322
82	112	217
83	108	278
84	98.6	379
85	91.0	270
86	81.4	418
87	81.0	348
88	53.4	395
89	47.9	526
90	40.4	588
91	39.8	339
92	35.0	1,700
93	19.3	442
94	17.9	444
95	16.4	526
96	15.4	1,490
97	10.8	425
98	7.4	550
99	3.59	3,760
100	2.2	1,880
101	0.81	6,050
	Colorado	
107	17,100	7
108	4,600	22.3
109	1,740	57.0
110	1,444	196
111	1,270	81.1
112	940	80
113	669	124
114	635	101
115	482	149
116	280	588
117	266	284
118	230	623
119	100	578
120	151	473
121	131	244
122	130	385
123	118	291
124	100	220
125	86	283
126	60	725
127	59.0	913
128	40	775
129	34.4	564
130	29.0	860
131	26.0	582
132	15.6	619
133	7.8	1,160
134	7.3	1,900
135	7.1	862
136	6.7	1,360
137	6.1	1,580
138	2.4	1,810
139	1.8	1,060
140	1.5	1,270
141	1.0	2,000
	Connecticut	
147	9,637	29.3
148	1,020	31.0
149	584	41.7
150	118	52
151	79.0	78
152	47.4	141
153	25.0	157
	District of Columbia	
159	11,500	42.0
160	77.5	126
161	62.2	71.7
	Florida	
167	1,220	28.2 (73.8)
168	336	89.3
	Georgia	
174	17,300	22

TABLE 1—*Continued*

Unusual Flood Peaks, United States and Foreign Rivers

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
175	7 304	48
176	5 180	{ 8 5
177		(20 0)
178	3 300	42 1
179	2 840	40 5
180	2 574	37 4
181	2 425	37 1
182	2 000	42 5
183	1 800	33 0
184	1 100	62
185	762	62
186	331	53
187	168	56

Idaho

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
193	41 900	1 13
194	22 600	2 40
195	13 400	9 0
196	5 480	9 4
197	4 850	16 8
198	2 230	9 9
199	1 670	10 7
200	1 220	18 0
201	1 080	16 0
202	960	7 9
203	940	10 5
204	717	15 1
205	586	15
206	420	21
207	131	32
208	5 0	1 000

Illinois

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
14	902 900	2 23
15	203 900	9 56
16	28 600	15 0
17	27 914	4 48
18	13 480	5 94
19	4 870	6 9
20	2 120	12 7
21	1 600	22 0
22	1 330	13 8
23	1 100	19 8
24	1 080	14 8
225	753	21 6
226	630	20 8
227	510	23 2
228	19 7	54 3

Indiana

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
234	11 300	20 8
235	4 940	27 5
236	3 760	30 0
237	295	23
238	160	35

Iowa

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
244	323 462	0 64
245	119 000	3 02
246	79 040	2 68
247	13 900	7 0
248	6 320	9 0
249	3 230	11 2
250	143	600
251	27 3	720
252	19 0	560
253	14 0	520

Kansas

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
259	59 841	3 80
260	44 910	4 0
261	24 960	6 7
262	9 490	9 13
263	3 670	20 3
264	3 067	16 4
265	2 0	910

Kentucky

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
271	921 900	(2 71)
272	202 700	9 1
273	90 600	12 1
274	60 600	12 2
275	7 500	27 7
276	6 170	16 0
277	4 890	23 6
278	3 320	26 0
279	2 150	33 1
280	2 010	29 6
281	0 260	127
282	982	40 8
283	746	48 8

Louisiana

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
289	1 400 000	1 07
290	1 243 700	1 61)
291	150 000	2 95

Maine

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
297	8 270	16 2
298	7 700	14 9
299	5 690	21 3
300	4 690	32 6
301	3 030	51 1
302	2 760	66
303	1 572	51
304	1 500	29 2
305	1 248	54 7
306	280	75 8

Maryland

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
312	9 654	50
313	875	97
314	817	79 2
315	302	83
316	287	261
317	247	177
318	191	133
319	165	118
320	148	182
321	94 4	239
322	39 0	290
323	36 1	285
324	31 3	311
325	5 7	790

Massachusetts

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
331	7 840	30 1
332	4 424	39 1
333	497	112
334	362	155
335	350	151
336	162	208
337	13 0	218
338	13 0	182

Michigan

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
344	4 900	10 1
345	2 461	20 1
346	800	13 4
347	112	17

Minnesota

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
353	36 800	2 01
354	17 100	2 87
355	14 600	3 0
356	12 400	4 1
357	5 950	6 0
358	452	40

Mississippi

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
364	1 144 500	2 18
365	13 850	10 0
366	1 400	42 8
367	650	{ 37 2
		(123)
368	15 0	1 110

Missouri

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
374	701 000	1 85

TABLE I—*Continued*

Unusual Flood Peaks, United States and Foreign Rivers

Item No.	Drainage Area, sq mi	Flood, sec-ft per sq mi
375	530,810	1.13
376	14,000	10.7
377	3,800	46.1
378	892	89.7
379	395	102
380	23.8	250
381	15.6	410
	Montana	
387	60,800	2.38
388	19,900	5.78
389	11,000	11.8
390	7,010	10.7
391	4,560	19.3
392	900	54
393	600	54
394	311	106
395	155	135
396	16	538
	Nebraska	
402	12,300	22.8
403	5,840	32.5
404	5,134	37
405	4,020	19
406	2,550	58.8
	Nevada	
412	13,800	0.22
413	2,150	3.8
414	1,070	7.0
415	414	13
416	298	10.4
417	10.0	17
	New Hampshire	
423	3,100	18.5
424	1,507	55.1

Item No.	Drainage Area, sq mi	Flood, sec-ft per sq mi
425	982	56.6
426	622	106
427	386	105
428	52	288
429	40	248
430	28	529
431	17.4	421
	New Jersey	
437	6,796	43
438	4,542	48.5
439	806	82
440	785	45.5
441	380	62
442	190	82.1
443	118	106
444	63.7	193
445	26	269
	New Mexico	
451	12,800	11.7
452	11,200	24.8
453	7,250	38.6
454	2,830	32.2
455	2,010	49.8
456	1,480	105
457	1,080	37.3
458	690	75
459	585	59
460	294	94
461	284	222
462	160	100
463	159	140
464	89	130
465	52	167
466	20.3	552
467	2.06	958
	New York	
473	298,080	1.07
474	263,440	1.13

Item No.	Drainage Area, sq mi	Flood, sec-ft per sq mi
475	175,000	1.71
476	8,100	27.2
477	4,500	26.7
478	3,456	40.6
479	3,076	50.4
480	2,530	36.5
481	2,240	28
482	2,150	41
483	2,055	67.2
484	1,492	55.5
485	1,370	44
486	992	44.0
487	900	55.1
488	897	46.0
489	538	110
490	783	68.1
491	770	54
492	735	60.8
493	593	77.6
494	472	80.2
495	467	64
496	444	56
497	417	132
498	372	105
499	344	102
500	339	75
501	264	75.8
502	241	79
503	236	123
504	222	90
505	210	100
506	193	286
507	186	136
508	185	135
509	124	208
510	101	165
511	100	267
512	89.2	207
513	71.5	173
514	59.4	448
515	56.7	742
516	49.8	442
517	46.1	657
518	35.8	391
519	29.6	547
520	21.3	1,310
521	21.2	424
522	20.8	726
523	18.1	320
524	18.0	466

Item No.	Drainage Area, sq mi	Flood, sec-ft per sq mi
525	16.5	721
526	12.1	825
527	11.5	1,550
528	11.0	585
529	9.5	966
530	7.04	952
531	6.41	1,040
532	5.0	654
533	2.91	2,520
534	2.49	1,130
535	1.68	1,150
536	1.5	2,300
537	0.62	835
538	0.25	3,300
	North Carolina	
544	8,350	32.9
545	6,910	30.7
546	4,290	31.0
547	3,930	46.8
548	1,600	50
549	1,340	73.7
550	949	94.8
551	675	85.3
552	662	58.2
553	150	90.7
554	97	211
555	41.4	170
556	27	1,110
557	22.0	1,340
558	12.2	1,350
558a	8.4	1,950
559	1.69	7,340
560	1.32	9,800
	North Dakota	
566	25,000	1.70
567	5,780	3.3
568	1,250	6.4
569	500	11.6
	Ohio	
575	75,800	12.5

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS UNITED STATES AND FOREIGN RIVERS

Item No	Drainage Area sq mi	Flood sec-ft per sq mi
576	7 410	37
577	3 937	94
578	3 850	65
579	2 510	100
580	1 6 4	84 9
581	1 570	70 5
582	1 195	69 4
583	1 130	113
584	1 047	80 8
585	649	117
586	646	132
587	514	98
588	448	115
589	270	244
590	65	66
591	5	571
592	11 8	1 280
593	6 7	2 [illegible]10
594	3 5	1 000
	Oklahoma	
600	96 900	[illegible] [illegible]8 (3 1)
601	108	640
602	61	560
603	42	860
604	41 5	1 370
605	37	1 450
606	4	1 590
	Oregon	
612	237 000	5 87
613	4 860	6[illegible] 3
614	1 450	64 3
615	204	[illegible]00
616	125	288
617	[illegible]0	1 900
	Pennsylvania	
623	26 800	32 5
624	19 106	34
625	18 300	29
626	11 220	21
627	9 010	33 8
628	7 707	24 1
629	7 [illegible]71	33
630	6 596	43
631	5 982	33
632	5 682	55
633	5 440	48 1
634	3 480	84
635	2 075	79 3
636	2 235	39
637	1 893	67
638	1 7[illegible]3	116
639	1 715	58
640	1 326	70
641	1 280	73
642	1 065	80
643	948	91
644	900	89
645	806	99
646	756	106
647	748	122
648	549	[illegible]
649	510	93 3
650	451	130
651	390	113
652	271	211
653	333	159
654	91	164
655	279	148
656	265	117
657	[illegible]1	154
658	210	143
659	200	185
660	187	154
661	153	183
662	62	1 000
663	48 0	580
664	22	410
665	20	750
666	13 9	1 000
667	7 0	550
668	2 2	1 630
669	2 3	1 120
670	2 1	1 090
671	1 7	1 690
672	0 67	2 540
673	0 60	4 000
674	0 6	760
675	0 6	493
676	0 58	4 170
	Rhode Island	
682	190	57
683	61	120
	South Carolina	
689	14 800	4 8
690	9 100	[illegible] 0
691	6 600	30 3
692	4 800	49 8
693	4 600	28 4
694	3 050	50 0
695	1 570	53 4
696	1 535	71 7
697	400	89
698	307	117
699	215	130
	South Dakota	
705	8 720	17 2
706	4 090	4 0
707	1 006	8 0
	Tennessee	
713	932 900	1 9
714	38 500	11 9
715	21 382	21 4
716	17 460	(22 9)
717	16 000	18 1
718	12 860	15 8
719	12 300	29 7
720	10 740	17 1
721	8 990	(27 8 21 7
722	7 320	20 9
723	4 446	(34 9)
724	8 000	25
725	3 060	(65 4) 23 2
726	2 470	(55 4) 47 8
727	2 297	30 5 23 2
728	2 100	84 8
729	1 940	30 4
730	1 880	53 0
731	1 870	37 4
732	1 860	(86 0) 33 4
733	1 640	128
734	1 210	(44 6) 30 2
735	1 180	46 7
736	1 140	64 5
737	857	53 3
738	795	54 0
739	793	190
740	764	(196) 88 6
741	692	(124) 57 8
742	675	85 2
743	624	191
744	552	81 5
745	452	77 4
746	439	79 4
747	353	90 7
748	312	224
749	97	170
750	48 7	540
751	46	315
752	12 5	490
753	12 0	810
754	10 7	680
755	10 2	390
756	8 4	790
757	6 6	2 470
758	4 2	1 630
759	3 2	1 090
760	0 4	11 200
761	0 17	1 900
	Texas	
767	128 218	4 9

TABLE 1—*Continued*

Unusual Flood Peaks, United States and Foreign Rivers

Item No.	Drainage Area, sq mi	Flood, sec-ft per sq mi
768	26,350	18.3
769	13,000	26
770	11,660	30.5
771	7,034	92.0
772	5,257	57.3
773	5,240	63.2
774	4,217	58.3
775	4,060	147
776	3,514	110
777	3,403	65.8
778	2,791	90.6
779	2,733	135
780	1,930	319
781	1,811	103
782	1,762	181
783	1,728	114
784	1,675	110
785	1,614	146
786	1,294	120
787	1,249	162
788	916	199
789	880	609
790	840	176
791	832	132
792	764	279
793	748	172
794	668	280
795	570	344
796	540	296
797	524	410
798	493	188
799	485	334
800	434	195
801	431	371
802	429	324
803	402	1,440
804	400	187
805	378	299
806	374	313
807	372	227
808	371	345
809	356	220
810	337	409
811	336	613
812	336	256
813	258	278
814	229	239
815	218	318
816	153	1,500
817	151	352

Item No.	Drainage Area, sq mi	Flood, sec-ft per sq mi
818	148	966
819	120	539
820	118	838
821	114	346
822	111	1,240
823	110	982
824	103	348
825	95	572
826	92.2	472
827	85.0	499
828	81	622
829	81	377
830	79	595
831	75.0	1,190
832	74.7	462
833	67	435
834	65.3	1,290
835	62.0	726
836	60.8	1,730
837	58.0	728
838	54.0	744
839	53	600
840	46.5	698
841	45.7	1,140
842	34.3	691
843	33	567
844	32	734
845	30	1,000
846	26.4	1,060
847	26	582
848	23.8	918
849	21.3	1,220
850	19.6	1,220
851	19	637
852	17.1	1,950
853	17	836
854	14	1,760
855	4.1	3,370
856	2.4	2,140
857	0.76	3,280
	Utah	
863	38,200	1.76
864	1,010	11.9
865	163	25
866	7.0	350
867	4.0	450

Item No.	Drainage Area, sq mi	Flood, sec-ft per sq mi
868	1.1	500
	Vermont	
874	4,068	33.4
875	1,070	108
876	690	174
876a	433	132
	Virginia	
882	6,757	23.4
883	3,080	24.4
884	2,730	29.6
885	2,725	63.8
886	2,084	44.2
887	1,638	69
888	1,599	41.3
889	831	44.8
890	388	72.0
891	331	65
892	304	95.2
893	208	95
	Washington	
899	70,000	7.03
900	24,200	8.98
901	5,520	11.5
902	3,550	18.0
903	1,020	25.3
904	1,170	43.5
905	500	51.2
906	202	93
907	184	200
908	136	100
909	105	238
	West Virginia	
915	37,950	17.1
916	23,800	21.3
917	8,376	32.2
918	5,936	56.5

Item No.	Drainage Area, sq mi	Flood, sec-ft per sq mi
919	3,040	49.4
920	2,430	38
921	1,471	97.2
922	1,380	116
923	1,344	46.5
924	1,304	57
925	1,185	(59.1)
926	1,145	79.7
927	972	67
928	759	(91.5)
929	719	118
930	686	134
931	670	154
932	458	(98.3)
933	393	104
934	271	159
935	230	109
936	200	150
936a	57.5	191
937	44	1,360
	Wisconsin	
943	45,000	2.98
944	10,300	7.84
945	8,000	10.0
946	6,740	9.0
947	5,800	16.1
948	5,600	13.9
949	2,780	16.2
950	675	34.2
	Wyoming	
956	20,700	1.97
957	8,080	3.7
958	6,050	15.7
959	794	61
960	520	61.5
961	72.0	97
	Foreign	
967	2,368,000	3.0

TABLE 1—*Continued*

UNUSUAL FLOOD PEAKS, UNITED STATES AND FOREIGN RIVERS

Item No	Drainage Area, sq mi	Flood, sec-ft per sq mi	Item No	Drainage Area, sq mi	Flood, sec-ft per sq mi	Item No	Drainage Area, sq mi	Flood, sec-ft per sq mi	Item No.	Drainage Area, sq mi	Flood, sec-ft per sq mi
968	1,945,000	3.5	976	4,100	239	984	336	410	992	10	2,010
969	1,100,000	2.73	977	3,360	74	985	178	694	993	5.3	2,070
970	367,970	4.9	978	1,320	98.9	986	58	1,380	994	4.5	1,600
971	149,800	12.7	979	862	493	987	52.5	667	995	1.5	1,600
972	86,620	5.3	980	841	382	988	42	1,300	996	1.1	2,810
973	62,000	6.9	981	587	324	989	23	1,950	997	1.0	3,250
974	58,000	10.6	982	544	432	990	20	1,750			
975	39,400	12.5	983	345	349	991	18.3	1,870			

When the enveloping curve is obtained, the next step is to determine whether the flood-peak-producing characteristics of the stream in question are as great as, or greater than, the maximum characteristics in that district, as indicated by that curve. Without evidence to the contrary it should be considered at least as great, the possibility that it might be greater always being recognized.

There are a number of significant physical characteristics that affect the peak-producing ability of a stream, and a comparison of them with those of the streams whose record floods influence the enveloping curve will often show whether or not a smaller flood peak than indicated by the enveloping curve may be used for the stream in question [2]. However, except where conditions are unusual, as in the case of extremely large natural lakes or swamps on the watershed, such comparisons are seldom made for preliminary studies, but the enveloping curve is used. In this connection, many earth dams with small ponds, having a spillway capacity much less than that indicated by such an enveloping curve, have been in existence for a number of years without damage. Although some have failed owing to overtopping, the fact that many more have not failed may be attributed to the circumstance that the record floods used to define the enveloping curve were all the result of a combination of conditions that made the probability of such an occurrence on a given watershed very remote. Data on such probabilities are greatly needed. On the other hand, there is the consideration that future enveloping curves for a given district will continue to rise as time passes and more records are obtained. These two factors tend to balance each other, and, for that reason, the use of the enveloping curve for preliminary studies is reasonable.

3. Effect of Physical Characteristics of Watershed. A knowledge of the physical characteristics of the watershed is helpful in evaluating its

flood-producing tendencies. Aside from differences in area of the watershed, two streams may have materially different flood tendencies, accounted for by a variance in the characteristics of the watershed. The flood coefficient C of Eqs. 2 and 4, which is made necessary by such differences in characteristics, depends on many factors, the chief of which are:

(*a*) *Storm Rainfall.* Intensity, areal distribution, orientation, direction, trend of great storms, effect of ocean and mountain ranges.

(*b*) *Infiltration.* Character of the soil, antecedent moisture, frozen ground.

(*c*) *Geographical Characteristics.* Shape and slope of watershed.

(*d*) *Natural Storage.* Valley storage, tributaries, lakes, and swamps.

(*e*) *Artificial Storage.* Reservoirs, channel improvements.

(*f*) *Land Coverage.* Forested, cultivated, pasture, and barren areas.

(*g*) *Sudden Releases of Flow.* Ice and log jams, débris jams against bridges, questionable safety of upstream dams, sudden snow melt.

The effect of areal distribution and orientation will be discussed later. If the general direction of storms is downstream, the tendency is for greater peak flows.

Storage, of whatever nature, tends to reduce the peaks of floods. The effect of lakes, swamps, ground storage, surface storage, and valley storage must be evaluated according to judgment and experience.

Deep sandy areas are conducive to large infiltration, therefore reduction of the peak and volume of floods. Antecedent moisture in the form of rain reduces infiltration. Frozen ground also materially reduces infiltration, particularly if the soil is fine-grained. The infiltration capacity of a coarse-grained soil may not be materially reduced by freezing.

Storage above the ground is affected by the nature of the vegetation, the shape and slope of the drainage area, and the characteristics of the river bed and banks. It is evident that those characteristics which will permit rapid runoff of the precipitation to the site of the dam will result in large floods. Rocky slopes devoid of vegetation are conducive to quick discharge. Conversely, areas covered with dense vegetation will prove effective in holding back the water and smoothing out the peak of the flood. Heavy grasses and underbrush are particularly effective in this respect. Forests also retard the melting of snow. At the peak of the precipitation practically no water is held back by adherence to leaves and branches of forests above the ground surface. For this reason many engineers in this country believe that it is the removal of the dense underbrush and surface humus rather than of the large trees that has increased flood tendencies in deforested districts. On the other hand, a commission of engineers studying floods in Germany has concluded that forests tend to mitigate the smaller and middle floods and that only in long and continued rainfall is this influence lost.

Cultivation, with furrows parallel to the surface contours, reduces peaks and increases infiltration.

Valley storage is the amount of water required to fill the river channel and valley to high-water elevation during floods. This sometimes has a tremendous influence on peak discharge. Frequent restriction in the river valley will tend to increase valley storage.

Steep slopes will produce rapid runoff. Therefore, floods from mountainous districts are relatively severe.

In rivers with tributaries extending in the shape of a fan from a given point, and of approximately the same size, the peak of the flood from each tributary is likely to reach the main stream and the dam at about the same time, resulting in relatively large floods. Conversely, when the catchment area is comparatively narrow, with tributaries of different sizes discharging into the main stream at regular intervals, the peak of the runoff from the tributary areas will reach the dam at different times, resulting in relatively small floods. A large number of tributaries also produces rapid runoff.

The creation of extremely long reservoirs materially decreases the time of runoff.

4. Flood-frequency Studies. The probable occurrence of all future *events*, based on an extended record of past occurrences, can be estimated according to the law of probabilities. In connection with water powers, such studies are useful in the determination of the probable frequency of the following events:

(*a*) Flood flows.
(*b*) Low river discharge.
(*c*) Depletion of storage reservoirs.
(*d*) Low annual rainfall.
(*e*) High rates of rainfall.

All such studies are based on the same basic principle as outlined herein for floods. The following flood-probability method was formerly used to determine the spillway design flood, but experience has demonstrated that it is not reliable and for such purposes it is no longer standard practice. However, it will be described here in a modified form * because it has the following important uses:

1. Special studies involving smaller frequencies, such as the probability of damaging the properties of the owner and relatively unimportant properties of others.
2. Special studies involving still smaller frequencies such as the probability of overtopping cofferdams.

The method involves the following steps:

1. The plotting of existing flood records in the form of a frequency probability curve, as in Fig. 2.
2. The extrapolation of this curve to intervals of time, I, greater than the length of the records. For example, in Fig. 2, a flood of 400,000 sec-ft may be expected on the average of once in $I = 200$ years.

* For full description see W. P. Creager, *Engineering for Masonry Dams*, 2nd ed., John Wiley & Sons, New York, 1929.

The law of probabilities may be applied to a study of flood frequencies by either of the following methods:

(*a*) *Basic-stage Method.* A consideration of all floods that exceeded a given basic stage during the period of records.

(*b*) *Yearly-flood Method.* The use of only the maximum flood during each year of record.

For the basic-stage method it is necessary to define what constitutes a flood. A single flood, as used in frequency studies, may be defined as an increase in flow above an assumed fixed basic stage, irrespective of the num-

TABLE 2

CALCULATIONS FOR PROBABILITY PLOTTING OF TENNESSEE RIVER

(1) Q Peak Flow	(2) Number of Occurrences	(3) Summation of Occurrences	(4) Calculated Percentage, p, of Future Floods	(1) Q Peak Flow	(2) Number of Occurrences	(3) Summation of Occurrences	(4) Calculated Percentage, p, of Future Floods
100,000	14	179	100.0	215,000	4	29	16.2
105,000	16	165	92.2	220,000	2	25	14.0
110,000	5	149	83.2	225,000	3	23	12.8
115,000	12	144	80.5	230,000	1	20	11.2
120,000	5	132	73.7	235,000	0	19	
125,000	6	127	71.0	240,000	1	19	10.6
130,000	12	121	67.6	245,000	0	18	. .
135,000	10	109	60.9	250,000	8	18	10.1
140,000	12	99	52.3	255,000	1	10	5.6
145,000	10	87	48.6	260,000	0	9	. .
150,000	3	77	43.0	265,000	3	9	5.0
155,000	5	74	41.3	270,000	1	6	3.3
160,000	7	69	68.6	275,000	0	5	. . . .
165,000	3	62	34.7	280,000	2	5	2.8
170,000	4	59	33.0	285,000	0	3	. . . .
175,000	3	55	30.7	305,000	1	3	1.7
180,000	3	52	29.1	310,000	0	2	. . .
185,000	2	49	27.4	345,000	1	2	1.1
190,000	7	47	26.2	350,000	0	1	. . . .
195,000	3	40	22.4	360,000	1	1	0.5
200,000	7	37	20.7	365,000	0	0	. . . .
205,000	0	30	. .				
210,000	1	30	16.8		$m = 179$		

ber of days the flow remained above that stage or the number of peaks and valleys in the flow before the discharge again receded below the basic stage

The choice of basic stage influences the number of floods to be considered, as an extremely high or an extremely low basic stage would obviously result in the consideration of very few floods Theoretically, the best basic stage is that which results in the largest number of floods, since by such means we have the greatest amount of data and hence the most accurate result Unfortunately however, in many rivers, the basic stage resulting in the greatest number of floods corresponds to a flow so low as to be entirely below the flood class, and some other method of fixing the proper basic stage must be devised

It is generally conceded that lake and swamp areas have a much greater influence on the frequency and magnitude of small floods than on those of large ones Consequently, it is becoming more generally recognized that in all flood-probability studies a greater weight should be attached to the larger floods It is therefore advisable to eliminate from consideration as many of the smaller floods as possible without reducing too greatly the number of floods included In general, a basic stage equal to or slightly lower than the lowest maximum yearly flood is recommended

An estimate of the probable frequency of flood flows of the Tennessee River at Chattanooga Tenn, made according to the law of probabilities is given below The method is strictly applicable to all similar problems

The discharge records of the Tennessee River that were considered extend over a period of 41 years—from 1875 to 1913 inclusive, plus 1916 and 1917 All floods* in excess of 100 000 sec-ft that occurred during those years are recorded in Col 2 of Table 2, which indicates the number of occurrences of floods with a magnitude between the corresponding flood in Col 1 and the next one below In this case 7 floods with a magnitude between 200 000 and 205 000 sec-ft occurred during the period of records

In Col 3 a summation is given of the occurrences indicated in Col 2 Column 3 therefore, shows the number of times, during the period a given flood was equaled or exceeded In this case a flood equal to or greater than 200,000 sec-ft occurred 37 times during the period

According to the law of probabilities the probable percentage of future floods that will equal or exceed a given flood, Q, may be obtained by the following equation

$$p = \frac{100n}{m} \qquad [5]$$

where p = the probable percentage of future floods that will equal or exceed a given flood, Q, expressed as a whole number,

n = the number of times, during the period of records, a flood, Q, was equaled or exceeded, as shown by Col 3,

m = the total number of floods that occurred during the period of records, in this case being 179 floods

* For this example the basic-stage method is used

This equation gives the values of p in Col. 4, which indicates that 20.7% of all future floods above 100,000 sec-ft probably will equal or exceed 200,000 sec-ft.

Values of flood discharge given in Col. 1 are plotted as ordinates, and percentages from Col. 4 as abscissas, on probability paper, as indicated in Fig. 2.

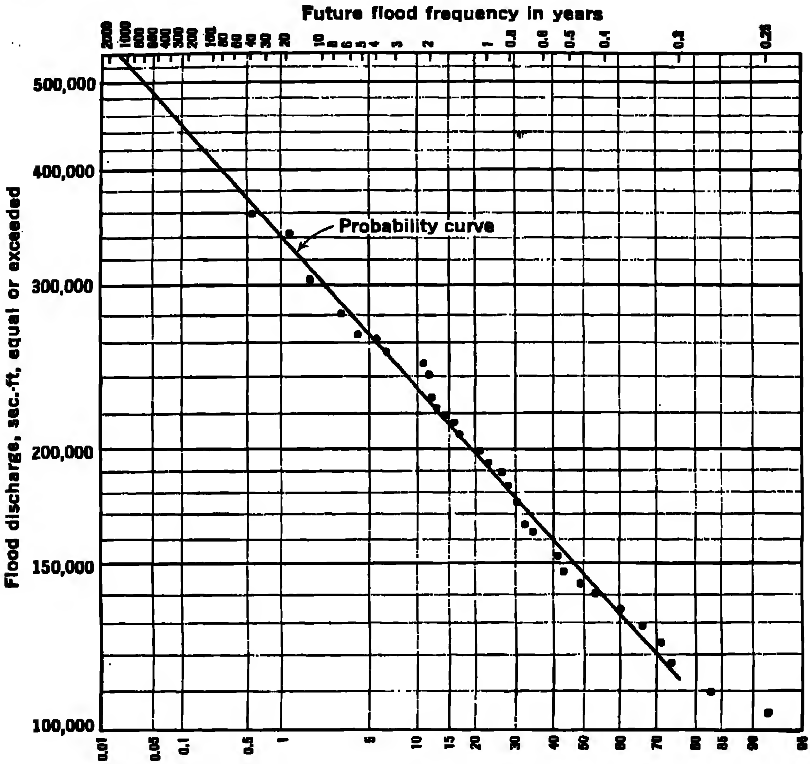

FIG. 2. Probability curve of Tennessee River at Chattanooga, Tenn.

A line is drawn through the plotted points and extrapolated to a limited degree, as previously explained, to obtain the probability of future floods in excess of the maximum of record. The line is not always straight as indicated in Fig. 2 but may be somewhat curved. A mathematical method is available for such extrapolation; * but, for the extent of extrapolation now considered allowable, an extension of the curve by eye is sufficiently accurate.

To obtain the frequency of occurrence, I, of future floods, let y equal the number of years of records, in this case 41 years. Then

* See p. 173 of *Hydro-Electric Handbook,* by Creager and Justin, John Wiley & Sons, 1927.

$$I = \frac{100y}{mp} \qquad [6]$$

For this example

$$I = \frac{100 \times 41}{179p} = \frac{22.9}{p}$$

By means of Eq. 6, the upper scale of frequency in Fig. 2 is calculated from the lower scale of percentage. The probable frequency of future floods can now be taken from Fig. 2, which indicates that a flood equal to or greater than 188,000 may be expected once each year and that a flood equal to or greater than 440,000 can be expected once in 200 years.

Great care should be exercised in the use of probability curves, particularly if the number of recorded events is not great. The frequency and magnitude of events during a relatively short period may correspond to a low or high cycle and therefore may not be indicative of average conditions. However, the use of probability curves, when limited as previously explained, is far preferable to the application of a factor of safety to the maximum recorded flood.

5. Physical Indications of Past Floods. Authentic federal and state government high-water records, extending over long periods, may be obtained for many streams.* Such records are often available also from mill operators and city officials. In general, however, the elevation of record high water must be determined from the observations and traditions of residents and from physical indications on the banks of the stream.

Observations and traditions of residents should be regarded with caution. Individual reports of untrained observers are subject to great error and are often of doubtful veracity, as the desire to report high water a little higher than reported by a neighbor is often, among certain classes, greater than the love of truth Unfortunately, also, reports are sometimes biased by a desire to give an impression of great or small floods, whichever in the opinion of the observer will better serve his interests. However, credence may be given when a number of observations closely agree and are referred to definite objects, such as sills of doors and windows or nails driven for reference.

Confirmation may be obtained from the elevation of deposited brush, logs, or alluvial matter, scars on banks and large trees from floating logs, and whatever other indications of high water may be discovered. High water, in a valley formed from the sediment from floods, is, of course, always higher than the surface of such deposits.

The elevation of record high water having been fixed, there are four methods by which an estimate of the corresponding discharge can be made.

1. The head on a dam which existed at the time of such high water can be determined, and from this the discharge over that structure can be computed by one of the well-known weir formulas.†

* U. S. Geol. Survey, U. S. Weather Bureau, Engineer Department, U. S. Army, etc.

† See Ref 15, Section 19, Chapter 8.

2. In unusual cases where the loss of head at contracted openings between bridge abutments has been observed, the approximate discharge can be computed from the expected loss at such openings.*

3. If a considerable stretch of straight river with a nearly uniform cross-section and slope is available, an estimate of the discharge can be made by the slope-area method, using Kutter's formula for the flow in open channels, particularly if accurate current-meter measurements of smaller floods have been made to determine the coefficient of roughness of the channel.

4. By the projection of a rating curve to the elevation of high water.† This method, however, is available only for a rough indication of the corresponding flood, unless the cross-section of the river is particularly regular and the discharge measurements used in plotting the curve cover a range including floods of considerable proportions.

Slope-area methods of determining discharge, mentioned in Item 3, are subject to three major uncertainties: (*a*) scour of the river bed during rising floods and subsequent refill as the flood recedes; (*b*) nonuniform rate of rise of water surface in the length of the stretch of river being gaged; (*c*) the choice of Kutter's coefficient, n.

(*a*) The beds of silt-laden rivers flowing on flat alluvial deposits may scour greatly because of the high velocities during floods and may subsequently refill as the flood recedes. For this reason it is necessary for accurate slope-area gagings to know the cross-sections, or at least to obtain a few soundings, at the peak of the flood. Measurements of the cross-section after the passing of the flood are worthless in some cases.

Sometimes deposits of immovable material located by borings, or other deposits which the geologist may classify as of ancient origin, will indicate the maximum possible scour that occurred during recent floods. Occasionally bridge piers that are known to have stood a flood without the aid of foundation piles will indicate by the elevation of their foundations the maximum scour that could have occurred.

(*b*) The slope of the water surface is steeper for the rising flood than for the highest water or the receding flood. Thus the maximum discharge may not occur when the water surface is highest.

(*c*) The best indication of the value of Kutter's n for use in slope-area gagings of a large flood is obtained from slope-area measurements in conjunction with some later, smaller flood that has been gaged by another method.

Values of n vary for natural channels generally between 0.025 and 0.035. However, floods overflow upon a different character of surface which sometimes may be considerably rougher and may even be wooded. For such surfaces the values of n frequently reach 0.040 to 0.060 and occasionally may be in excess of 0.100.

* Section 11, Chapter 8.

† A rating curve shows the relation between gage height and discharge as indicated by discharge measurements.

Photographs of river types with recommendations for values of *n* will be found in "Flow of Water in Drainage Channels," by C. E. Ramser, *U. S. Dept. Agr. Tech. Bull.* 129, November 1929. See also "Some Better Kutter's Formula Coefficients," by R. E. Horton, *Eng. News*, Feb. 24, 1916, p. 373.

6. Storm Rainfall. Records of major storm rainfall can be found in the publications of the U. S. Weather Bureau. Isolated records have been kept also by local public and private organizations, the most comprehensive of which is that of the Miami Conservancy District [3]. Bailey and Schneider [4] have summarized the Miami and other data and have prepared charts giving probable excessive rainfall in eastern United States.

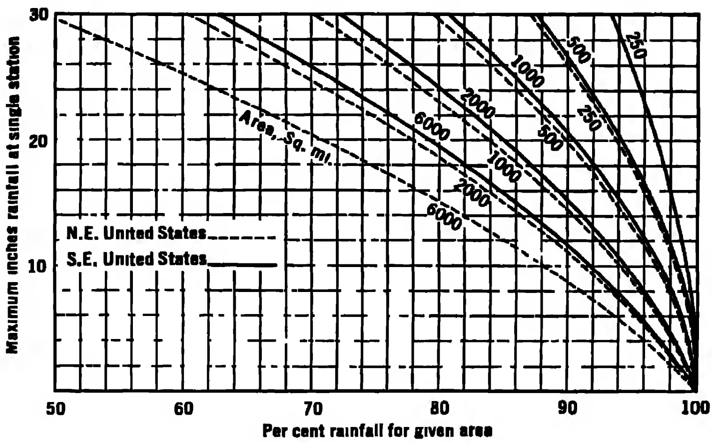

FIG. 3. Average rainfall for given area in percentage of maximum rainfall at single station

The War Department, Office of the Chief of Engineers, Washington, D. C., has started the accumulation of comprehensive data necessary for evaluating flood potentialities of drainage basins. A large part of this study has been completed. It includes not only rainfall at single stations but also variations with drainage area and mass rainfall curves.

The relation between the rainfall at a single station and the average rainfall over a given area surrounding that station varies with each storm. For this reason a safe value must be used to obtain the average rainfall on a given area derived from the records at a single station. Figure 3 gives these values based upon enveloping curves of data contained in the Miami reports and the War Department studies.

All published records of average storm rainfall and average storm rainfall derived from Fig. 3 are bounded by equal isohyetal lines which, in all probability, resemble in no way the shape of the area tributary to the dam.

It must be remembered that the area in question will have the same rainfall as that of the area within the isohyetal lines of the storm, only (*a*) if the

storm is centered exactly on that area, (*b*) if the storm is of the same pattern as the area, and (*c*) if the storm is oriented exactly on the area. On the other hand, any record storm for a given district is bound to be exceeded in the future. Some engineers believe that the chances of a record storm centering on, shaping to, and orienting on a given drainage area are much less than the chances of the record storm being exceeded. To this opinion is attributed the fact that many earth dams in this country are still standing with spillway capacities much less than dictated by the methods of the more radical engineers.

The choice of the magnitude of the storm rainfall to be used for flood studies is therefore a question of probabilities about which very little is known. It is, then, more a matter of judgment and precedence than an exact analysis.

7. Storm Runoff—Volume in Flood. Studies based on infiltration rates lead to a reasonable estimate of the percentage of rainfall that appears as runoff during the period of the flood. Such studies are quite involved and are seldom used in preliminary investigations.

It has been demonstrated that the capacity of a given soil to absorb rainfall falling continuously at an excessive rate rapidly decreases until a fairly definite minimum rate of infiltration is reached, usually within a period of a few hours. Therefore, the percentage of rainfall that appears as runoff increases materially with the intensity and duration of the storm. It approaches 100% in some localities. Conservatism is necessary in this respect since the available data seldom correspond to a storm which sufficiently approaches that assumed for the spillway design flood. When good rainfall records are available, the plotting of runoff percentages for storms of several intensities may allow a fair amount of extrapolation for the spillway design storm.

Winter storms on frozen ground without a snow cover are conducive to high runoff. With snow on the ground a runoff greater than 100% might be expected and may be equal to the runoff plus the water equivalent of the snow cover. However, the insulating effect of deep snow has a tendency to reduce the frozen condition of the soil.

Usually 8 to 10 in. of fresh snow is the equivalent of 1 in. of water. However, 10 in. of old snow may be the equivalent of as much as 5 or 6 in. of water.

Owing to the many variables and complex factors affecting snow melt during major storms, very little practical information is available. For preliminary studies it may be assumed that the maximum snow-melt runoff may be 0.1 in. per degree-day of temperature above 32 degrees Fahrenheit, limited, of course, to the amount of snow mantle. For instance, if the average temperature is 42 degrees during a 4-day storm, the degree-days above 32 degrees Fahrenheit would be 40 and the runoff would be 0.1 times 40 or 4 in.

8. The Shape of the Hydrograph. The approximate shape of the flood hydrograph can be obtained by the following method:

The peak flow having been estimated from Section 2 and the total storm runoff from Section 7, the next step is to determine the shape of the hydrograph. The length of the base of the hydrograph may be determined by

$$D_a = \frac{MAi}{Q} \tag{7}$$

where D_a = the length of base of hydrograph in days;
A = the drainage area in square miles;
i = the total runoff from the watershed in inches;
Q = the peak discharge in second-feet;
M = a constant obtained from Fig. 4.

Let A = 270 sq mi, i = 11.1 in., and Q = 60,000 sec-ft. From Eq. 7

$$D_a = \frac{120 \times 270 \times 11.1}{60{,}000} = 6.0 \text{ days}$$

Figure 5 shows dimensionless hydrographs that are accurate enough for preliminary studies. For a given drainage area (270 sq mi in the example), vari-

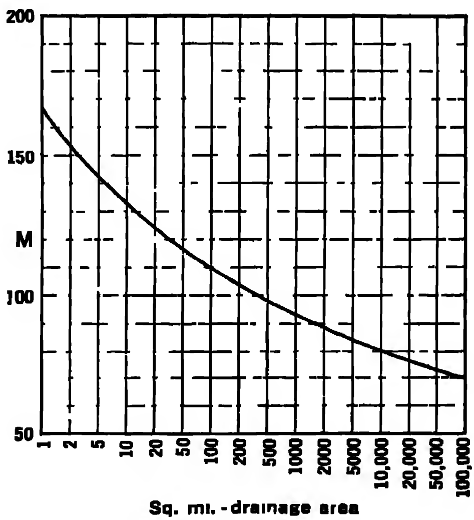

Fig. 4. Values of M.

ous values of D, less than the total flood period D_a, are chosen and D/D_a is calculated. From Fig. 5, the corresponding discharge ratios Q'/Q are obtained, from which the discharge Q' at any time D, after the beginning of the flood, can be calculated and plotted in Fig. 6 for the inflow of the flood.

9. Routing Floods. The surcharge storage in large reservoirs, above the spillway crest, has a material influence in reducing the peak of the flood, as shown in Fig. 6.

The flood of the foregoing example is routed, as indicated in Fig. 7, which illustrates a convenient method [5] of routing floods through a reservoir. In this figure, the curve Q is the spillway rating curve. Curve S is the sur-

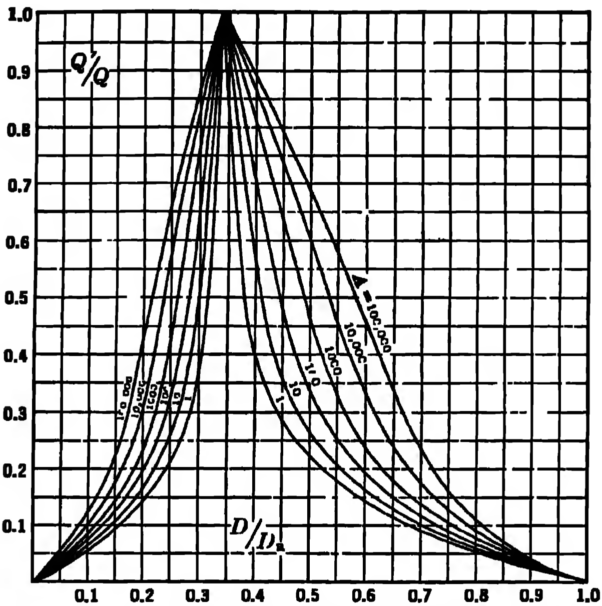

FIG 5 Dimensionless storm hydrographs for various drainage areas.

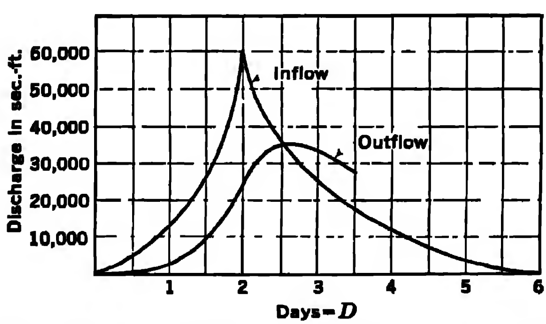

FIG. 6. Inflow and outflow hydrographs for the given example.

charge storage in the reservoir. The spillway crest is at Elev. 430.0. When the flood rises to Elev. 437.1 the reservoir stores 40,000 acre-ft of water.

The routing computations require a step-by-step method for certain intervals of time as indicated in Table 3. For this example an interval of time of 12 hr was used. The smaller the interval of time, the more accurate are

the computations Usually a smaller interval of time than that shown in Table 3 is required to obtain reasonable accuracy.

Curve *B* is obtained by subtracting from Curve *S* one half the spillway discharge capacity at corresponding elevations of water surface, expressed in acre-feet per 12 hr For this example, the spillway discharge with water

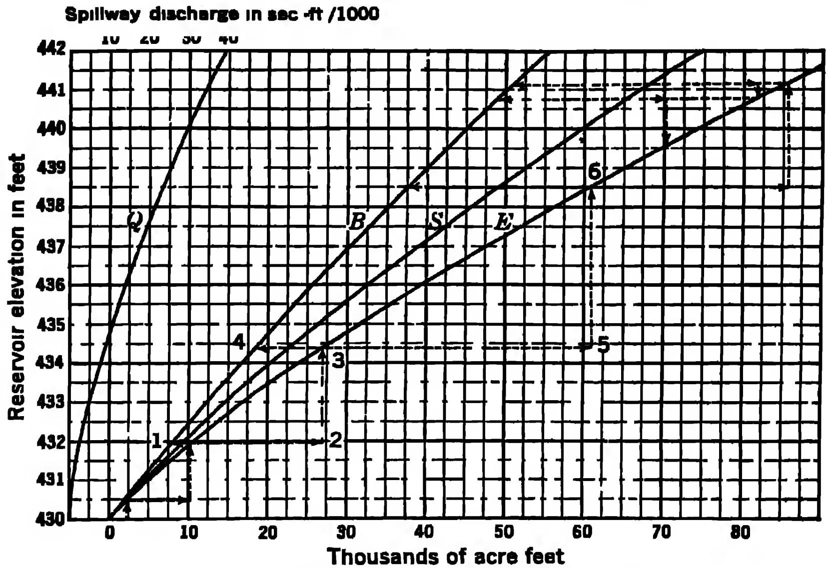

Fig 7 Inflow-storage-discharge (I S D) curves

surface at Elev 434 8 is 10 000 sec-ft and the corresponding discharge, expressed in acre-feet per 12 hr, is

$$\text{Acre-ft} = \frac{\text{Second-feet} \times \text{Hours}}{12\ 1} \qquad [8]$$

$$\text{Acre-ft} = \frac{10{,}000 \times 12}{12\ 1} = 9930$$

Therefore, at Elev 434 8, a point on Curve *B* is plotted by subtracting one half of the discharge capacity or 9930/2 4965 acre-ft from Curve *S*

Similarly, Curve *E* is obtained by adding one half of the corresponding acre-feet to Curve *S*.

The routing procedure is illustrated in Fig 7 and Table 3 Columns 2 to 4 of the table contain data on inflow rates and volumes corresponding to the hydrograph of Fig 6 The first steps in the process are obscure because of the small volumes of inflow involved, but the procedure is similar to that indicated by the series 1, 2 3 (Fig 7) pertaining to the third step of the routing

TABLE 3

SAMPLE FLOOD-ROUTING COMPUTATIONS

Step	(1) Time from Beginning of Flood, in hours	(2) Instantaneous Rate of Inflow into Reservoir (I), in c.f.s.	(3) Sum of Discharges at Beginning and End of Interval ($I_1 + I_2$)	(4) Volume of Inflow into Reservoir during Interval,* in acre-feet	(5) Reservoir Elevation at End of Interval	(6) Spillway Discharge Rate at End of Interval, in c.f.s.
1	12	4,500	4,500	2,230	430.5	500
2	24	12,500	17,000	8,440	431.9	2,600
3	36	26,000	38,500	19,100	434.4	8,800
4	48	60,000	86,000	42,650	438.5	23,800
5	60	37,000	97,000	48,100	441.2	35,000
6	72	25,500	62,500	31,000	440.7	33,000
7	84	18,000	43,500	21,580	439.5	28,000

* Average rate of inflow during interval = $\frac{1}{2}(I_1 + I_2)$ (approximate).

The 36-hr increment of inflow (19,100 acre-ft) is laid off as abscissas 1-2 from Curve B at Elev. 431.9, the elevation of water surface at the end of the second step, and a vertical line is drawn from 2 to an intersection with Curve E at 3, to obtain a reservoir elevation (434.4 ft) at the end of the third step or the 36th hour of the flood. The fourth increment of flow (42,650 acre-ft) is then laid off from Curve B as abscissas 4–5 and a vertical line is again projected from 5 to 6 on Curve E to obtain the reservoir level (438.5 ft) at the end of the fourth step or the 48th hour of the flood. In this manner the entire reservoir-stage hydrograph tabulated in Col. 5 of Table 3 is computed. The rates of outflow in Col. 6, corresponding to various reservoir levels, are read from Curve Q. The results are plotted in Fig. 6.

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CHAPTER 7

INVESTIGATION OF SITES

1. Scope. The investigations necessary for the sites of the intake, conduits, surge tank, and powerhouse are of the same general nature as those required for dams. A thorough investigation to determine the most desirable location of each structure should precede the design and construction of the project.* Of the structures appurtenant to such a project, the dam is generally the most important. Although the preliminary investigation of the whole project comes within the scope of this chapter, emphasis is placed on the location of the dam site.

2. General. The investigation of a site usually requires: (*a*) topographic mapping, (*b*) geologic studies, (*c*) subsurface explorations, and (*d*) investigation of available construction materials, such as earth and gravel and concrete aggregates.

3. Extent of Investigation. The advisable extent of such investigations depends in part on the magnitude of the project and in part on how obvious the surface and subsurface conditions are. Thus the amount of time and money devoted to investigation of a site where the entire foundation consists of solid granite, water-worn and exposed to view, would probably be much less than that required at a site in a valley having a deep accumulation of overburden on stratified and folded rock of unknown quality. Similarly, it is seldom necessary to make as extensive an investigation for a project having a low diversion dam as for one having a high earth or masonry dam.

Occasionally the investigation of sites for hydroelectric projects has been overly thorough and expensive. On the other hand, it is frequently difficult to get an owner to allow a sufficiently thorough investigation prior to the design and construction. Consequently trouble and excessive cost have often resulted from insufficient preliminary investigation.

4. Reconnaissance. Serious examination should be preceded by considerable reconnaissance work and rough figuring. The engineer, when he first goes on the ground, should have some idea of the sort of site or sites for which he is looking. He should know how much power is desired and have an open mind, ready to adopt that type of development which best fits the local conditions of topography, geology, and market. The dam should be

* Any proposed scheme for obtaining electrical power from falling water is called a hydroelectric project herein. When completed, it becomes a hydroelectric development.

located as close to the turbines as possible, thus providing the maximum size of pond and the shortest length of conduit.

The reconnaissance should involve visiting all possible sites which are available for consideration, and gathering such information relative to such sites as is obtainable without subsurface exploration. The best maps available should be obtained. In many sections, the government topographic maps, made by the U. S. Geological Survey, are extremely useful to engineers on reconnaissance work. Aneroid barometers, hand levels, a metallic tape, and a camera are usually desirable equipment. A relatively brief office study with the data gathered in the reconnaissance will usually result in the elimination of all but a few of the sites.

5. Preliminary Investigation. The preliminary investigation usually requires:

(*a*) A not too precise stadia site survey with the resulting topographic site map

(*b*) Some investigation of the overburden.

(*c*) A few borings, say from six to fifty, according to the magnitude of the project and the character of the foundation.

(*d*) A preliminary geologic investigation and report.

(*e*) Investigation of available construction materials, such as earth and gravel and concrete aggregates.

(*f*) The determination of public utilities which the project might affect, such as roads, bridges, railroads, telephone and telegraph lines, pipe lines, and power plants.

(*g*) In the relocation of the above facilities, a fairly accurate topographic map of the basin is essential.

(*h*) Hydrologic studies.

(*i*) The checking of high-water marks and their use in determining spillway capacity requirements.

(*j*) Reservoir evaporation studies.

The objective of the preliminary investigation is to obtain only sufficiently precise data to permit office studies and estimates of cost of enough accuracy to determine the most economical and suitable site among the several selected by the reconnaissance survey.

A consideration which, for storage dams, affects the choice of general location is the quantity of silt carried by the stream. In some streams this is enormous and may in the course of a few years fill the reservoir *sufficiently* to destroy its usefulness for storage.

6. Final Investigation. After the preliminary investigations at the several sites have been made and office studies and estimates for each of them completed, one of the several sites is selected for final, precise investigation. The site survey and the resulting topographic maps should be sufficiently accurate and exact to serve all the purposes of construction. All necessary borings, test pits, subsurface explorations, geologic studies, and tests on the materials in the foundation and in the proposed borrow pits will be made.

As a result of the final investigation the engineer should have available all the pertinent data to proceed with the detailed designs of the structures and the making of a control estimate of cost for construction.

The line of demarkation between preliminary and final investigation of a site is not sharp. One often blends into the other. The point is that in the early stages of investigation, where several sites are involved, the amount of investigation should be limited to that necessary to determine the relative merits of the sites, thus avoiding the possibility of making a precise and costly investigation only to find that subsurface conditions are such that the site will have to be abandoned in favor of one of the alternative sites.

The final investigations are usually supervised by the engineer who has conducted the preliminary examinations or are at least conducted in accordance with his recommendations. The principal aims are:

(a) To determine the relative merits of two or more sites for the project in question so that a final location can be adopted.
(b) To determine the type of dam to be used.
(c) To settle beyond a doubt, by subsurface investigations, the nature of the foundation as affecting the safety and cost of the dam and other structures.
(d) To fix the limits of the lands to be controlled for flowage, for the sites of structures, and for other necessary purposes.
(e) To determine the extent and character of relocation of railroads and public highways necessary on account of raising the water surface.
(f) To ascertain the character of the government regulations to be observed.
(g) To obtain sufficient information for an accurate estimate of cost.
(h) To determine the final location of the dam, intake, conduits, surge tank, powerhouse, construction equipment, camps, cofferdams, construction highways, and railroads, as well as the probable source of materials of construction and all other information needful to the constructing engineer.
(i) To obtain all necessary information affecting the design of the project.

7. Choice of Location. In the choice of location several structures may be involved, such as the dam, intake, conduits, surge tank, and powerhouse, which are to be combined into a scheme for development. There is no easy answer as to the choice of location of appurtenant structures for any particular arrangement. The only sensible way is to make several alternative layouts and to adopt the one which appears to be most economical. Sometimes the most desirable location is perfectly apparent to an experienced engineer. Sometimes there is little opportunity for choice, and considerable investigation and study are required to determine which is the most economical location.

Once the general location is chosen, the exact position of the appurtenant structures will be fixed after careful consideration of the following factors:

(a) Requirements as to head, flow demands, and storage capacity.
(b) The character of the foundation, particularly for the dam.
(c) The topography of the surface at the proposed location.
(d) Availability and character of construction materials.
(e) The value of the necessary lands and water rights.
(f) Arrangement and type of dam, intake, conduits, surge tank, powerhouse, and tailrace.
(g) Transportation facilities and accessibility of the site.
(h) Availability of suitable sites for construction, equipment, and camps.
(i) The cost of the project.

8. Topographic Mapping. Topographic maps for preliminary investigations are produced either by rough stadia or plane table methods, or by the application of aerial photographs, the last being the most popular for large areas and difficult terrain. Aerial photographs can very readily produce sufficiently large topographic maps showing features and conditions which can be advantageously compared for alternative sites. Topographic maps from aerial photographs are made by stereoscopic restitution methods. The methods vary depending upon the plotting equipment used and the accuracy and contour interval desired.

For final investigations and layouts, more precise topographic maps are required. Such maps are generally prepared from stadia surveys.

9. Geologic Investigations. For the investigation of dam sites and conduit sites the employment of a geologist experienced in this work is essential.

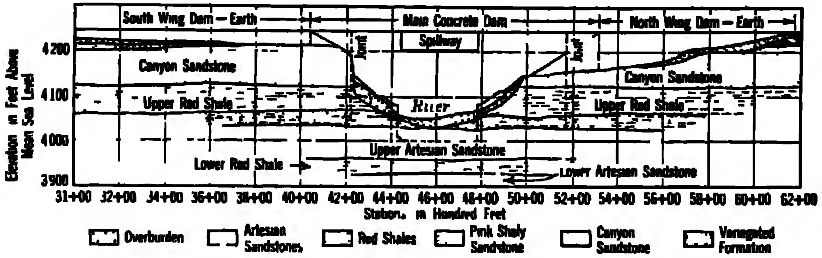

FIG. 1. Geologic section across Conchas Dam, New Mexico. (Irving B. Crosby, *Trans. A.S.C.E.*, Vol. 105, p. 583, 1940.)

All good geologists have extensive imaginations, and the engineer never ceases to marvel at the completeness of the geologic report, which a geologist is often able to prepare after a few days of scouting around a site. The data may not be very precise but will be extremely helpful in indicating where the drilling had best be done.

In fact it should be an invariable rule never to start a drilling program in work of importance without the advice of an experienced geologist. Slides, faults, folds, sink holes, dips, and strikes may carry implications to a geologist which entirely escape the engineer. For instance, a limestone which is interbedded with strata of shale and which has not been subject to extensive fracturing or faulting is much less likely to have extensive solution channels or cavities than one which has such fractures and does not have the interbedding of shales.

Some sites have been extensively and expensively prospected with diamond drilling only to find unsatisfactory foundation conditions; whereas, if a good geologist had been engaged beforehand, he could have told the engineer in advance approximately what he would find and could have directed him to another site where prospects of success were more propitious. In the interpretation of the cores from the borings, the geologist can also be of material assistance. On work of magnitude it is quite usual to utilize the services of a

geologist throughout the period of investigation and also to engage the services of a particularly experienced geologist as a consultant.

As the work of subsurface exploration continues, geologic sections at and near the site are drawn up with the help of the geologist. Such geologic sections are very useful to the engineer in studying his foundation problems and his grouting program. In Fig. 1 is shown a geologic section at the site of Conchas Dam, New Mexico. Geologic sections greatly facilitate the study of foundation conditions. They should never be used as information for bidders or as a part of the contract plans for the reason that they necessarily involve the use of imagination, and the engineer may thus lay himself open to the charge of misrepresentation. Only the cores of the borings themselves and the carefully interpreted log of the holes should be utilized as information for bidders.

10. Subsurface Exploration. Extensive subsurface explorations are usually necessary in order to aid the geologist and the project engineer to determine foundation conditions and to investigate available construction materials. By far the most satisfactory way to explore the underground is to dig test pits, shafts, and tunnels. Get down there, examine the material, take samples, and test them. Ordinarily, if such methods were depended on exclusively, the cost of thorough subsurface exploration would frequently be prohibitive. Consequently it is necessary to rely on methods which are supplementary or alternative, such as electrical prospecting, seismic exploration, and borings of various kinds.

11. Electrical Resistivity and Seismic Methods of Prospecting. In some cases the engineer is not greatly interested in the nature of the overburden above the ledge. What he wants to know is how far it is to rock and what the character of the rock is when it has been reached. In such cases, after a few widely scattered boreholes have been placed to serve as a basis for reference, either the electrical or seismic method of prospecting may be used to advantage. Anyone who thinks that these methods can take the place of diamond drilling may receive a bitter disappointment.

The methods are supplementary and can be used to develop general surface features of the ledge and thus indicate where it is advisable to place additional holes through the overburden and diamond-drill holes into the rock. If used in this manner, with a full understanding of their limitations, they may lead both to a material economy in the cost of subsurface exploration and a better knowledge of subsurface conditions.

The electrical-resistivity method is dependent on the difference in conductivity of the water contained in the ledge rock. The seismic method requires the measurement of the rates of propagation of waves caused by explosions and is dependent on the difference in the elastic properties of the ledge rock and the overburden. Both these methods necessitate trained and experienced personnel. For most engineers interested in foundation work it is sufficient to know the conditions under which the methods might be useful. Then, if the engineer decides that one or both of these methods would be useful to

him, he should engage a specialist to make the exploration. Under some conditions one method is superior to the other, and consequently there is an advantage in engaging a specialist who is familiar with both methods.

12. Wash Borings. Wash boring and churn drilling are inexpensive methods of getting through the overburden, but they should not be depended on for showing the character of material passed through.

In wash boring a small pipe is used on the inside of a larger pipe or casing. At the end of the inside pipe is a nozzle, sometimes with a chopping bit. Water under pressure is admitted to the inside pipe and jiggled up and down, with the result that a mixture of water and material is discharged over the rim of the casing. This loosens the casing at the lower end, and the casing is then driven down a little farther, usually by a weight operated from a tripod. The down-driving is continued, with ledge rock as the desired depth to be reached.

By catching the mixture of water and material as it comes out of the casing, one may obtain samples of a portion of the material passed through which is both disturbed and washed. By carefully watching the wash water all the time one can sometimes get a fair idea of whether he is passing through sand and gravel, sand, or sand with silt and clay, etc. All in all, wash boring is usually the cheapest way to get through the overburden; it is the poorest way to learn anything very definite about the character of the overburden.

13. Churn Drilling. In churn drilling, a barrel or spoon is utilized inside the outer casing. The spoon or barrel is of limited length, and the method, instead of using a superabundance of water, as in wash boring, uses only a relatively small amount of water in the casing. Sometimes the required water is furnished entirely by the natural water in the underground. The barrel or spoon has a flap valve or tail valve at or near its lower end, arranged so that material may enter the barrel but cannot get out. A chopping bit of varied design is usually placed below the entrance to the barrel. The barrel and chopping bit are then jiggled up and down by means of a rope or cable attached to their upper end and the material is thus forced into the barrel. The casing is pounded down as the operation of the spoon continues. When the barrel is nearly full, it is removed and the material inside it dumped out. Most of the material passed through is present in the sample, but it is mixed up, partly segregated, washed, and just about as far as possible from being in its natural condition. A power-operated earth auger is also provided with some drilling rigs; it usually affords an inexpensive way of obtaining disturbed samples of the overburden.

The above is the usual well-drilling method. By the use of heavy equipment and special chopping bits, the method has been developed for penetrating even relatively hard ledge rock. Both wash-boring and churn-drilling methods permit taking drive samples at intervals.

An inexpensive method of subsurface investigation of overburden is furnished by the use of hand-operated earth augers ("Iwan" and other types). Samples are disturbed, but the method may answer for most of the samples

if one is investigating a borrow pit, for instance. Depths of 20 to 40 ft may be reached in some soils by this method.

14. Rotary Drilling through Overburden. Rotary core drilling is similar in principle to diamond drilling. Using some form of hardened-steel core bit with cutting edge, the driller may put down exploratory holes through some firm clays and compact sands and silts without casing the hole and may obtain a core of the material. Drilling fluid, consisting of a rather thick mud formed from fine clay or bentonite, is kept in the hole, and the rotating bit presses it into the walls of the hole, thus giving the walls sufficient strength to stand up. The core is also smeared with the drilling fluid, but the impregnation is usually slight. The writer has seen compact sand cores obtained in this manner in which the drilling fluid impregnated less than 1/16 in. into the sample. The method was developed in the oil fields but has also been used quite successfully in exploration work in materials such as those described above. If diamond drilling is to continue after rock is reached, the hole through the overburden is cased before the diamond drilling is started, as usually the walls cannot be relied on to stand up indefinitely. The use of the rotary drilling rigs for the exploration of overburden material has its limitations.

In very stiff clays the rotary drill is often useful. Excellent undisturbed samples of Trinity sand, a fine dense sand whose void ratio may be as low as 0.21, were obtained at the site of the Denison Dam, Texas and Oklahoma. The method is not successful for sampling coarse sands and gravels.

15. Undisturbed Sampling of Overburden. In order that the character, strength, and permeability of the overburden may be determined, it is necessary to obtain undisturbed samples of the material and put them through a series of tests in the laboratory. Such tests on undisturbed samples are particularly necessary for clays, silts, and very fine sands. It is usually of less importance to make tests of undisturbed samples of coarse sands and gravels because their stability or shear strength is usually adequate and permeability may usually be determined within the necessary range of accuracy by making tests on disturbed samples at approximately the same density as the material possessed before disturbance (see Chapter 16).

Test pits furnish one of the best means of obtaining samples which are for all practical purposes undisturbed. The only disadvantage is the expense of excavating at depth. At shallow depths, say 5 or 6 ft, the test-pit method is as cheap as any other. For the cost of one deep test pit, say 60 ft, however, one could take a large number of undisturbed or nearly undisturbed samples by drilling and driving methods.

To take an undisturbed sample of clay in a test pit, a small area in the bottom is carefully leveled off. Then the four sides of the sample, usually approximately 8 by 12 in., are carefully cut or carved and the surrounding material is removed. The five sides of the sample are then paraffined and wrapped with two or three thicknesses of cheesecloth as the paraffin is applied. Then when all is ready the container is fitted over the top of the specimen

and a sharp spade severs the specimen from its pedestal. The sample is turned over, and its top, which is the bottom in its natural position, is paraffined and covered with fabric. The packing is now completed, and the container is labeled. The specimen is ready to go to the laboratory and may, if necessary, be stored for a considerable period of time before tests are made.

16. Diamond Drilling. For drilling exploratory holes in ledge rock the diamond core drill is almost invariably the most suitable tool. A core of the strata passed through is obtained which may be studied by the geologist and the engineer so that a true picture of the foundation, including the presence of faults, seams, cavities, and changes in the quality of the rock, may be secured.

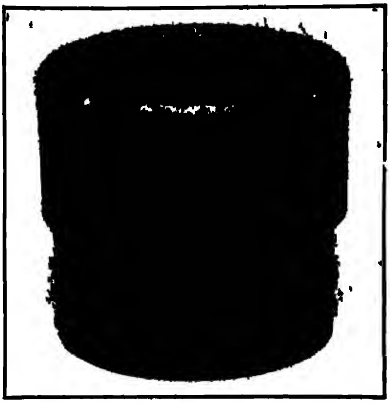

FIG. 2. Cast-set (bort) diamond bit. (Sprague & Henwood.)

The diamond drill is rotated by means of a gasoline, steam, or Diesel engine or an electric or air motor. At the head of the drilling machine is a screw feed or hydraulic feed for maintaining just the desired amount of pressure on the drill rods. There is also a hoist for raising and lowering the drill rods, and a tripod is usually set up over the hole to facilitate this. The drill rods are of special, heavy construction with flush joints.

The cutting edge, as shown in Fig. 2, is a steel ring set with black diamonds commonly called "carbon," or with an impure crystalline form of diamond known as "bort." Above the ring, or diamond bit as it is commonly called, is a core barrel into which the core passes as the bit cuts down into the rock. At the bottom of the core barrel is a split-ring spring core lifter, by means of which, when the column of drill rods, core barrel, and bit are lifted out, the core will be caught and retained. When the assembly is in action, water is pumped down on the inside of the drill rods to keep the diamond bit cool and to carry the rock cuttings up on the outside of the drill rods. When a double core barrel is used, the inner tube of the core barrel fits down over the core, and its upper end is plugged so that water from the inside of the drill rods passes down between the inner and outer tubes of the core barrel and does not touch the core at all except at the cutting edge of the bit. This is because it has been found that the mere presence of water under high pressure tends to destroy the cores of the softer rock. It is good practice to use the double tube core barrel in exploratory work in all except the very hardest and most massive rocks as higher percentages of recovery are thereby promoted.

17. Importance of Accurate Boring Data. The importance of accurate boring data is indicated by the following incident. Some exploratory work

for a tunnel showed the strata to consist of hard quartzite interbedded with a soft greasy shale. In the diamond drilling an "EX" bit (about the smallest commercial size) was used, giving a core of about ⅞ in. in diameter. A single core barrel was used. As a result the soft greasy shale was almost entirely ground up and washed away so that only the quartzite was recovered. The cores were placed in boxes without spacers to indicate the missing strata. The actual recovery was approximately 65%, but practically all the quartzite was recovered.

The contract drawings followed the apparent indications of the core boxes and showed massive and continuous quartzite. The core boxes and the drawings mentioned constituted the information available to bidders.

The information was grossly misleading, and the successful bidder eventually recovered a substantial sum representing the additional cost of driving a tunnel through the difficult formation encountered over what it would have been with sound firm rock, which the information available led him to expect.

Many incidents similar to the above, where misleading data relative to exploration work have resulted in costly extras, might be cited. They emphasize the importance of thorough exploratory work, properly interpreted. However, the main reason for sparing no pains to obtain as true a picture of subsurface conditions as practicable is that only in that way can the engineer design his structures with assurance of both safety and economy.

A high percentage of recovery is greatly to be desired, as it decreases the amount of guessing which is necessary. If the total length of core recovered equals the depth of the hole in the rock, the recovery is said to be 100%. In many rocks, by the use of proper methods and equipment with proper diameter of bit, 100% recovery is not only possible but is frequently approached. In actual practice, 80 to 90% recovery is quite common.

Cores should be kept in suitable core boxes and properly labeled and described in every respect. They should be at least 2⅛ in. in diameter. Smaller cores are misleading and worthless.

18. Large Drill Holes. In the exploration of a dam site it has become rather common practice to put down one or more 30- to 36-in.-diameter holes generally drilled by shot drilling methods. These holes have the very great advantage that they leave no doubt at all about the character of the rock passed through. The engineer or geologist can have himself lowered to the bottom of the hole in a boatswain's chair or in a cage operated from a hoist and can study every inch of the walls. A true picture of all the strata passed through is thus obtained. The only objection to these holes is their cost, which is usually \$40 to \$70 per ft, or 15 to 40 times as much as ordinary diamond-drill holes. In tunnel exploration, however, they are a satisfactory and economical substitute for a shaft in ledge rock.

19. Rough Methods of Determining Suitability of Materials. When one is making a reconnaissance investigation of a proposed site, the facilities of a well-equipped laboratory are often not readily available. Nevertheless

it is necessary by a process of elimination to select the most promising locations for the structures, borrow pits, or quarries before adopting intensive methods of investigation. By observation and very simple rough tests, it is practicable to determine in an approximate and tentative manner the suitability of material for a concrete dam, a rolled fill dam, or a hydraulic fill dam, as well as for other structures.

If, in looking for a quarry site for concrete aggregate or riprap, one finds an exposed cliff where the rock is hard and firm and does not tend to break into thin laminations when pounded with the hammer, and there is no evidence of easy weathering, the indications are favorable for a rock suitable for concrete aggregate and for riprap, and further investigations and tests should be made.

If the sand and gravel is well graded and appears to be largely quartz or fragments of hard igneous or metamorphic rock it is probably suitable for concrete aggregate, although if dirty it will have to be washed. Chert (amorphous quartz) is not necessarily objectionable provided it is hard.

Such sand and gravel would also be suitable for the pervious portion of an earth dam provided they did not contain too much very fine material.

If sand and gravel, when dropped into a bucket of water and sloshed around, leave the water very muddy, the material will probably have to be washed before it will be suitable for concrete aggregate.

If a cohesive borrow-pit material intended for the impervious portion of an earth dam is taken in the hands, kneaded, and then rolled out to a diameter about that of an ordinary pencil and shows up just slightly crumbly, then the moisture content is not too great to prevent proper compacting by means of suitable equipment. The above is merely the rough form of the plastic limit test. It is usually easy to add water to such material if the material is not sufficiently moist, but too high a moisture content in such material may be serious.

If the material consists of sand and gravel with sufficient clay or rock flour to make it sticky when wet, and if when packed and dished it holds water for a long time, it will be suitable for the impervious portion of a rolled fill dam.

Practically all sandy and gravelly materials are suitable for hydraulic fill dams provided only that the fine material included does not contain too much colloidal material.

A powerful field microscope is useful in helping to determine in a qualitative manner the suitability of some materials. Such microscopes, having a magnifying power of 200 diameters or more, are now obtainable. They are made to collapse into a small container readily carried in the pocket. At 200 diameters a particle of rock flour having a diameter of 0.005 mm looks like a sizable rock. Particles which at this magnification look gluey and have no definite crystalline form are probably largely colloidal and have diameters of less than 0.002 mm.

By rough field tests and observations, such as the above, an experienced engineer can usually determine in a tentative manner the suitability of the

available materials. As the choice of materials and sites narrows down, laboratory tests should be applied more intensively.

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CHAPTER 8

HYDRAULICS

1. Flow through Orifices and Short Tubes. Typical examples of orifices and short tubes are given in Fig 1. A knowledge of the laws of the flow of water through them is necessary in determining the discharge through sluiceways and the entrances to conduits. If the entrance is not properly shaped, a contraction of the jet occurs as in Sketches *a*, *c*, and *h* and the area of the jet is not as great as the area of the orifice or tube. For properly rounded approaches to orifices, as in Sketches *b* and *e*, and in the constant-

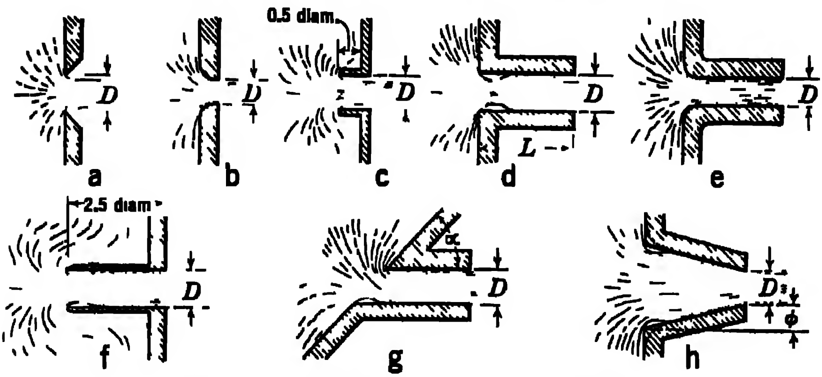


Fig 1. Examples of orifices and short tubes

diameter short tubes shown in Sketches *d*, *f*, and *g*, the diameter of the jet is equal to the area of the orifice or tube. In the case of short tubes without rounded entrances, the contraction does occur, but the jet expands again* as indicated, a partial vacuum occurring just inside the entrance.

Let H = the head of water in feet on the center line of a freely flowing orifice or tube, or the difference in water level for a submerged orifice or tube,

a = the area in square feet of the orifice or tube,

v = the theoretical velocity in feet per second corresponding to head H;

g = the acceleration of gravity = 32.2 ft per second per second;

Q = the discharge in cubic feet per second;

C'' = the coefficient of contraction, or ratio of the area of the jet to the area of the orifice or tube;

C' = the coefficient of friction;

C = the coefficient of discharge.

* With certain exceptions as explained later

The general equation for the velocity of spouting water is

$$v = \sqrt{2gH} \qquad [1]$$

Considering friction, the actual velocity due to head H is

The discharge is equal to the product of the actual velocity and the area of the jet, or, since the area of the jet is $C'''a$,

$$Q = vC'''a$$

or

$$Q = C''C'''a\sqrt{2gH}$$

In experiments conducted to determine the discharge through oriffices and tubes, the coefficient of friction, C'', and the coefficient of contraction, C''', are combined, and the general equation for discharge is

$$Q = Ca\sqrt{2gH} \qquad [2]$$

The value of the coefficient of discharge, C, varies with the shape of the orifice or tube. Average experimental values of the coefficient of discharge, C, for use in Eq. 2, for several types of orifices and tubes are given in Table 1.* These orifices and tubes are circular unless otherwise described. According to Bovey and other authorities on orifices in thin plates (Fig. 1a), the value of C for circular orifices is about 2% less than for square orifices; from 3 to 4% less than for rectangular orifices with a ratio of length to height of 4; and from 5 to 7% less than for rectangular orifices with a ratio of length to height of 10. A similar relation probably exists for tubes with square-cornered entrances.

The coefficient C is *not greatly* affected by submergence.

Coefficients for short tubes apply only to heads less than about 40 ft. For higher heads the expansion previously explained does not occur and the coefficients C approach those for orifices of similar type.

The expansion of the jet within the short tubes will not occur if they are not submerged and if sufficient friction is lacking In other words, the jet, after contracting, will shoot through the tube without touching its sides, if under a high head; and even if the trajectory is such that the jet strikes the bottom of the tube, the expansion will not occur if the friction along the bottom is insufficient.

The shape of the jet from a sharp-cornered round orifice (Fig 1a) between the orifice and the *vena contracta*, or section at which the area of the jet becomes constant, is indicated in Fig. 2† and should be used for rounded entrances as in Fig. 1e.

* Head large in comparison with size of orifice or tube. See later discussion
† A. H. Gibson after Weisbach.

TABLE 1

COEFFICIENTS OF DISCHARGE, C, THROUGH ORIFICES AND TUBES FOR EQ. 2. CIRCULAR EXCEPT AS NOTED

Figure	Type	Coefficient C
1*a*.	Orifice in thin plate	0.60
1*b*.	Bell-mouth orifice (Fig. 2)	0.97
1*c*.	Inwardly projecting orifice	0.50
1*d*.	Short tubes with sharp-cornered entrances,*	
	Values of L/D	
	0	0.60
	0.25	0.63
	0.50	0.67
	0.75	0.72
	1.00	0.76
	1.50	0.79
	2.50	0.80
	3.50	0.80
1*e*.	Short tube with bell-mouth entrance (Fig. 2)	0.97
1*e*.	Short tube with rounded entrance	0.90
1*f*.	Inwardly projecting tube with sharp-cornered entrance	0.72 to 0.80
1*g*.	Inclined short tube with sharp-cornered entrance,†	
	Values of α	
	90°	0.82
	80°	0.80
	70°	0.78
	60°	0.76
	50°	0.75
	40°	0.73
	30	0.72
1*h*.	Convergent short tube,‡ sharp-cornered entrance	
	Values of ϕ	
	0°	0.82
	5.75°	0.94
	11.25°	0.92
	22.50°	0.85
1*h*.	Round-cornered entrance	
	Values of ϕ	
	0°	0.97
	5.75°	0.95
	11.25°	0.92
	22.50°	0.88
	45.00°	0.75

* From experiments by Rogers and Smith on submerged tubes, *Eng. News*, Vol. 76, p. 827, 1916. The coefficient for L/D of 2.5 and greater has been found by other experimenters to be 0.82.

† According to Weisbach.

‡ H. W. King after Unwin.

The coefficients in Table 1 have been obtained from experiments under ideal conditions. As an indication of the smaller discharge coefficients obtained for practical cases, Fig. 3 gives the results of Stewart's experiments [2] on the discharge through 4.0-ft-square submerged sluices of different types.

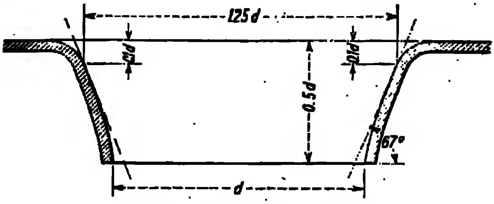


FIG. 2. Bell-mouth entrance.

The minimum value of C for each set of experiments is shown. The forms of the entrances to the sluices are given below and are shown in Fig. 4.

Series A, square entrance;

Series a, contraction suppressed on bottom;

Series b, contraction suppressed on bottom and one side;

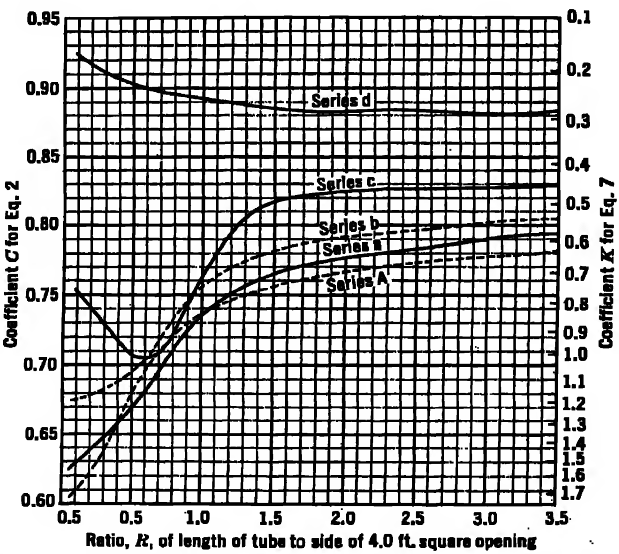


FIG. 3. Minimum coefficient of discharge, C, and corresponding coefficients, K, from Stewart's experiments.

Series c, contraction suppressed on bottom and two sides;

Series d, contraction suppressed on bottom, two sides, and top.

The experiments covered heads ranging from 0.05 to 0.30 ft. It was found that the coefficient C was least for heads of 0.15 ft, and these losses are the ones indicated in Fig. 3.

In these experiments, Series *A* corresponds to Item 1*d* of Table 1, in which the coefficient *C* varied from 0.60 to 0.80 as compared with Stewart's coefficients of 0.60 to 0.784, which is a reasonably close agreement; but Series *d* corresponds to Item 1*e* of Table 1, in which *C* is given as 0.97, as compared with Stewart's variation of 0.925 to 0.882. In the latter case, however, the considerably lower values of Stewart's coefficient were probably caused in part by an imperfect mouthpiece.

Velocity of Approach. If the velocity in the channel of approach were uniform, the actual head on the orifice or tube would in reality be the measured head plus the head corresponding to the velocity of approach. The velocity of approach, however, is not uniform through any section of the channel, being less than the average at the sides and bottom. The velocity directly opposite the orifice is therefore greater than the average, and the effective head on the orifice is the measured head plus a head somewhat greater than that corresponding to the velocity of approach.

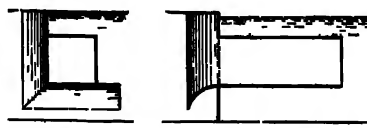


FIG. 4. Stewart's sluice, series *b* (showing contraction suppressed on bottom and one side).

Equation 2, corrected for velocity of approach, may be written

$$Q = Ca\sqrt{2g\left(H + \beta\frac{v^2}{2g}\right)} = Ca\sqrt{2g(H + \beta h_v)} \qquad [3]$$

where v is the average velocity of approach, h_v the head corresponding to this velocity, and β a coefficient which must be determined experimentally. Unfortunately, β is not well known for many types of orifices and may vary between 1.0 and 2.0, depending on the location and relative size of the orifice.

Rectangular Orifices and Tubes under Low Head. If the area of the freely flowing orifice or tube is large in comparison with the head, the equation for discharge should be written as follows:

$$Q = \frac{2LC}{3}\sqrt{2g}(H_1^{3/2} - H_2^{3/2}) \qquad [4]$$

where H_1 and H_2 are the heads on the bottom and top of the orifice, respectively.

If the orifice is completely submerged, the head is the difference in level between the upper and lower water surfaces, and Eq. 2 applies.

If in Eq. 4 the top of the orifice is at the water surface, $H_2 = 0$ and the equation reduces to

$$Q = \frac{2LC}{3}\sqrt{2g}H^{3/2} \qquad [5]$$

This is the basic theoretical equation for discharge over weirs.

Gibson says that Eq. 2 can be used with an error not greater than 1.0% if the head on the top of the freely flowing orifice is greater than twice the height of the orifice.

2. Discharge through Sluice Gates. Sluice gates are made in a variety of forms. Many types of sluices, controlled by sluice gates, are in reality short conduits in which skin friction is a large percentage of the total loss. A discussion of the losses through conduits is given later. If the sluice is short, the discharge can be considered as that through a short tube (Figs. 1*d* to 1*h*, inclusive), or, if the sluice gate is in a thin wall (Figs. 1*a* to 1*c*, inclusive), the discharge can be obtained from Eqs. 2 or 3 with the proper coefficient of discharge selected from Table 1 or Fig. 3, according to the details of the sluice-gate opening as explained in Section 1.

3. Loss of Head in Conduits. The loss of head in conduits can be divided into two general groups:

(*a*) Eddy losses, which are caused by sudden changes in the direction of flow, as at bends, branches, etc., or by sudden changes in velocity due to sudden changes in area as at the entrance, sudden enlargements, valves, etc.

(*b*) Skin friction in straight, uniform conduits.

4. Eddy Losses in Conduits. It is convenient to measure eddy losses in terms of the velocity head of the flowing water. The velocity head, or head required to produce a given velocity, can be obtained by transposing Eq. 1 of Section 1:

$$\text{Velocity head} = h_v = \frac{v^2}{2g} \tag{6}$$

Then h_f, the head lost at any point in the conduit due to eddies, is

$$\text{Eddy loss} = h_f = Kh_v = K\frac{v^2}{2g} \tag{7}$$

where K is the coefficient of eddy loss and v is the velocity in the smaller section unless otherwise defined.

5. Losses at Conduit Entrances. A discussion of the flow through short tubes and sluices is given in Section 1, and the coefficients of discharge C for several types of entrances are indicated. There is a direct relation between the coefficient of discharge for short tubes and the coefficient of eddy loss, K (Eq. 7), which can be derived as follows:

A drop in pressure or head at the entrance to a conduit is required for two purposes:

(*a*) Velocity head to provide the necessary velocity;

(*b*) Head to overcome friction due to eddies.

Or,

$$H = h_v + h_f = \frac{v^2}{2g} + K\frac{v^2}{2g} = \frac{v^2}{2g}(1 + K)$$

As $Q = av$,

$$H = \frac{Q^2}{2ga^2}(1 + K)$$

Also from Eq. 2,

$$H = \frac{Q^2}{2ga^2} \cdot \frac{1}{C'^2}$$

Combining, we have,

$$K = \frac{1}{C'^2} - 1 \qquad [8]$$

The loss of head in short tubes where there is no residual contraction of the jet, as in Figs. 1*d*, 1*e*, 1*f*, and 1*g* and in the sluices used in Stewart's experiments given in Section 1, may be assumed to be the same for similar entrances to closed conduits; and values of K derived from the experimental values of C' of Section 1 are given below.

TABLE 2

COEFFICIENTS OF EDDY LOSS FOR EQ. 7

Figure	Type	Coefficient K
1*d*.	Short tube with sharp-cornered entrance	0.56
1*e*	Short tube with rounded entrance	0.60
1*f*.	Inwardly projecting tube with sharp-cornered entrance *	0.56 to 0.93
1*g*.	Inclined tube with sharp-cornered entrance	
	$\alpha = 90°$	0.49
	$\alpha = 80°$	0.56
	$\alpha = 70°$	0.65
	$\alpha = 60°$	0.73
	$\alpha = 50°$	0.78
	$\alpha = 40°$	0.88
	$\alpha = 30°$	0.93

* Depending on distance of projection.

Values of K for the sluices used in Stewart's experiments, which are also applicable to losses at entrances of similar type, are given in Fig. 3 and discussed in Section 1.

6. Losses at Conduit Intakes. Losses at the intake racks are usually very small because the velocity must be low to facilitate raking. For clear racks, a safe approximate value of K for use in Eq. 7 can be obtained from the following equation:

$$K = 1.45 - 0.45R - R^2 \qquad [9]$$

where R is the ratio of net to gross area of racks and supports, and v in Eq. 7 is the velocity in the net area of racks and supports. For a usual value

of $R = 0.05$ the resulting value of K is 0.74, and for a usual velocity of 2.5 ft per second in the net area, the loss is 0.072 ft. However, allowance should be made for partial closure of the racks with trash. Twenty-five to fifty per cent of the area of hand-raked racks is frequently obstructed in practical operation where the amount of débris in the water is considerable. This would increase the loss in the above example 0.13 to 0.29 ft.

The head losses in well-designed intakes, with the exception of the loss at the racks and gates, are usually negligible, as changes in area are gradual.

Losses at the head gates correspond to the losses previously given for conduit entrances, the worst practical condition being that in which the gate is at the head of an entrance similar to the one indicated in Fig. 1*d*. Usually, however, it is possible to provide economically a flare at the upper end of the intake, so as to produce a gradually increasing velocity between the racks and the gates, with sides and bottom of the intake nearly flush with the gate opening. In such cases there is a decided eddy only at the top of the gate, and the coefficient K is no greater than about 0.5 in Eq. 7, where v is the velocity through the gate.

The gradual increase in velocity between the gate and the normal section of the conduit is made without appreciable loss.

Intakes to open flumes vary greatly in type according to the conditions to be met and the ideas of the designer. Losses in well-designed open flume intakes are usually negligible.

For data on losses in intakes see Refs. 3 and 12 of Section 19.

7. Losses at Conduit Bends. In the present discussion, values of K for use in Eq 7 in deriving the losses in bends of conduits are such as to

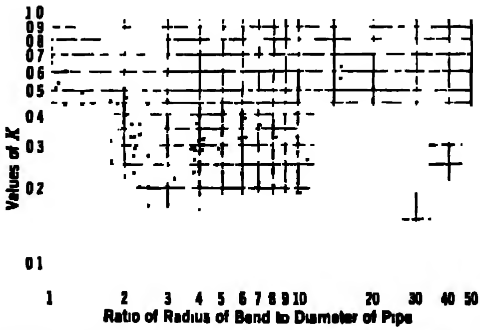


FIG. 5 Experimental values of friction coefficient in 90° pipe bends.

give the loss in excess of that which would occur in a straight pipe of equal length. In other words, normal skin friction is not included.

An effort has been made in Fig. 5 to show the conclusions of a number of experimenters regarding the value of K for 90-degree bends in closed conduits. The result shows an extreme lack of agreement, which leads to the

conclusion that the loss is not independent of the kind of pipe used in the test. The following discussion would seem to substantiate this conclusion.

The loss of head in bends in excess of that in a straight pipe of equal length is probably caused by two predominating conditions: namely, increased skin friction and eddy losses.

Investigations seem to indicate that, owing to centrifugal force, the flow must adjust itself in the bend to a greater ratio of maximum to average velocity. As skin friction varies approximately as the square of the velocity,

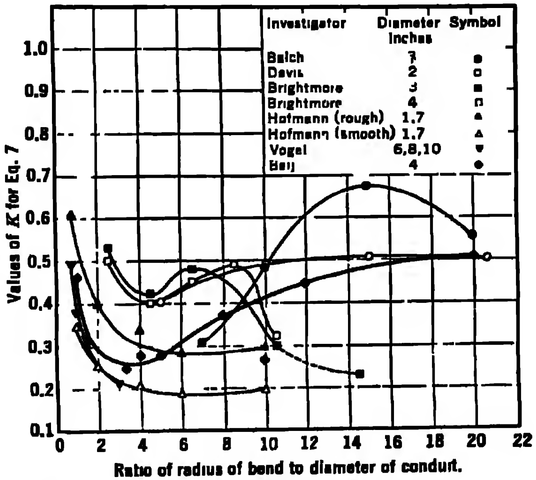


FIG 6. Bend coefficients found by various investigators (K. H. Beij, "Pressure Losses for Fluid Flow in 90° Pipe Bends," *Natl. Bur. Standards Research Paper* RP 1110, July 1938, p. 10.)

the skin friction would naturally be greater than in a straight conduit, and hence this loss is dependent on the length of the bend and the type of conduit.

Eddy losses are caused by an adjustment of the stream lines at the entrance and at the exit of the bend. These losses are probably independent of the length of the bend.

Experiments for closed conduits have tended to show that bends with a ratio of radius of bend to diameter of conduit equal to about 5 correspond to the minimum loss in the bend, and hence to the minimum value of K, as indicated in Fig. 6.

This bend ratio of 5 may be the critical value where the eddies caused by the adjustment of stream lines at the entrance to the bend are somewhat smoothed out by overlapping the eddies caused by the adjustment of stream lines at the exit of the bend. In other words, it is conceivable that the eddies at entrance and exit could tend to synchronize or partially counteract each other for a bend ratio of a certain value.

Although experimental data are meager, the results, as presented in Fig. 6, indicate two flow regimes in 90-degree conduit bends. For very sharp bends, the coefficient decreases rapidly to a minimum at the critical bend ratio of about 5. As the bend ratio is still further increased, the coefficient depends not only on the flow, the bend ratio, and the roughness of the conduit, but also on some sort of instability, as shown by the wide range of results.

Figure 7 gives recommended values of K for 90-degree bends in closed conduits, based on available data. The curve representing safe values is an enveloping curve for all experiments, and the curve representing probable

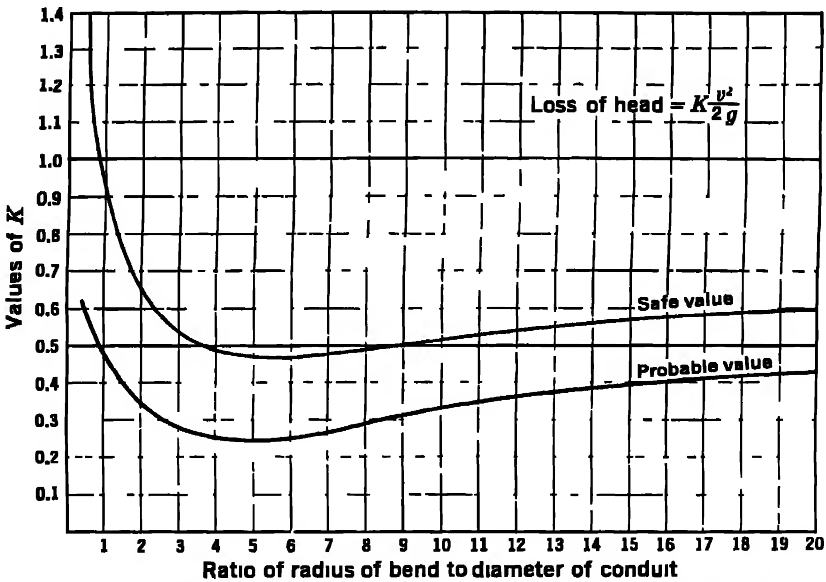


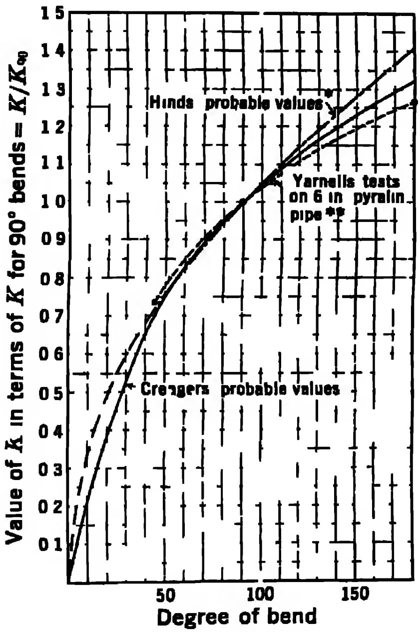
FIG. 7 Recommended values of K for 90° bends in closed conduits

values is the mean of the experiments. Fifty per cent should be added to the values of K from Fig 7 for screwed pipe elbows on account of the sudden enlargement and contraction in such fittings.

It is recommended that values of K for bends other than 90 degrees be taken from Fig. 8.

Few experiments have been made on the losses in bends of open conduits. Fortunately, sharp bends in open conduits are not usually required. Engineers sometimes allow for loss at bends of open conduits by adjusting the coefficient, n, of the flow equation. Frequently a coefficient is given for a straight open conduit and another coefficient for the same type of conduit "with moderate curvature" or some other equally indefinite description. The authors claim that an allowance for bends should not be made in the flow coefficient, n, but that the slope for straight alinement should be determined

first, and increased slopes provided at bends corresponding to the best judgment regarding the additional losses at the bends. The few experiments made seem to indicate that the loss due to bends in open conduits is much less than that for closed conduits, and values equal to one half of those in Fig. 7 are recommended. The authors have compared such values with experimental data and have found them to agree very closely when one half the "probable curve" value is used.



* See p 1033 of Ref 5 of Section 19
** See p 1042 of Ref 5 of Section 19

Fig. 8. Value of K for various degree bends in terms of K for 90° bends.

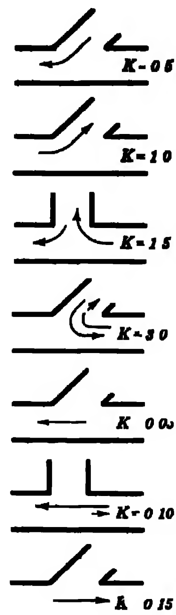


Fig. 9. Recommended safe values of K for miscellaneous fittings.

For reverse bends in both open and closed conduits the loss is probably somewhat greater than if the two bends were separated by a considerable length of straight pipe but the amount of such loss has not been determined.

Recommended safe values of K for miscellaneous fittings are given in Fig. 9. These values are for all the flow in one direction. For uniting or dividing flow in such fittings the reader may consult Ref. 6 of Section 19.

8. Losses at Conduit Valves. The value of K for use in Eq. 7 for wide-open gate valves is probably less than 0.1, although few experiments have been reported.

Dow's experiments on the disk-arm type of butterfly valves (Fig. 34, Chapter 27) indicate a value of K equal to

$$K = \frac{t}{d} \qquad [10]$$

where t = the thickness of the disk of the valve, and
d = the diameter of the valve.

The value of v for use in Eq. 7 is that in the normal section of the conduit. For the coefficient of loss for needle or plunger valves (Fig. 23, Chapter 26, and Fig. 8, Chapter 28), the following equation is presented:

$$K = \frac{0.183}{\sqrt[3]{d}} \qquad [11]$$

where d is the diameter at the small end, in feet. The value of v for use in Eq. 7 is that in the small end of the valve. Results from Eq. 11, compared with experimental data, are as follows:

Source	d	v	Experimental Feet Loss	Experimental K	K from Eq. 11
Larner Eng. Co.	1.5	20.0	1.00	0.16	0.16
Wellman, Seaver, Morgan Co.	4.67	19.0	0.65	0.116	0.11
Larner Eng. Co.	12.0	20.0	0.50	0.08	0.08

Equation 11 applies only to needle valves in closed conduits. For free outlets, as in Fig. 8, Chapter 28, the loss due to the valve can be determined by Eq. 2. In this case a is the small area of the valve in square feet and C is the coefficient of discharge, varying from 0.64 to 0.76 depending on the make of the valve.

Equation 2 also applies to the discharge of a Howell-Bunger valve at the end of a pipe, using a coefficient, C, of 0.9 and an area, a, equal to the area of the pipe.

9. Miscellaneous Conduit Losses. As all losses other than skin friction and bends are due to sudden changes in section of the conduit, a knowledge of the laws governing the losses due to sudden contractions and enlargements will assist materially in determining the losses in various irregularities of conduits.

The theoretical value of K (for use in Eq. 7) for a sudden enlargement in section of a closed conduit is

$$K = \left(1 - \frac{a_1}{a_2}\right)^2 \qquad [12]$$

where a_1 and a_2 are the areas of the smaller and larger sections, respectively. The value of v for use in Eq. 7 is that in the smaller section. Actual losses follow the theoretical losses fairly closely. Losses due to gradual enlargements in a closed conduit are not known exactly. Etcheverry gives the value of K as follows:

$$K = \left(1 - \frac{a_1}{a_2}\right)^2 \sin\theta \qquad [13]$$

where θ is the angle between the axis of the pipe and the surface of the pipe.

Equations 12 and 13 are frequently used to determine the value of K for sudden enlargements in open conduits and for the junction of a closed and open conduit, but incorrectly so, as the fundamental principles used in their derivation do not apply to open-water conditions. The value of K should be taken from the following equation: *

$$K = \left(1 - \frac{a_1^2}{a_2^2}\right) C \qquad [14]$$

where the coefficient C has a value dependent on the nature of the enlargement. Scobey † recommends the following values for the coefficient C:

For square enlargements	$C = 0.75$
For enlargements with each side set at a 30-degree angle with the flume axis	$C = 0.50$
For perfectly designed enlargement transitions [3]	$C = 0.25$

The value of v for use in Eq. 7 is that in the smaller section.

For sudden contractions in closed conduits, the values of K are given in Fig. 10, which was derived in a manner similar to that employed by Merri-

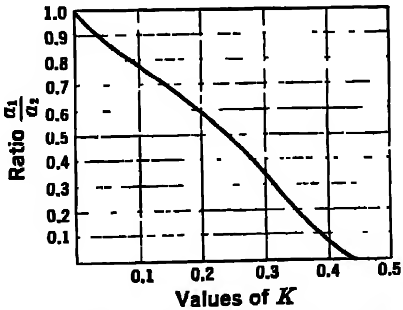


FIG. 10. Values of K for sudden contractions in closed conduits

man, except that the coefficient of contraction for a sharp-edged orifice, equal to 0.60, was used instead of 0.62, which he employed in his basic equation. The value of v for use in Eq. 7 is that in the smaller section. Losses due to gradual contractions are very small.

The loss of head in sudden contractions in open conduits is covered in the explanation of the application of Eq. 31a of Section 11 to the discharge through restricted channels. The loss in gradual contractions of open conduits is negligible.

* The exception is where the hydraulic jump is introduced.

† See p. 15 of Ref. 12 of Section 19.

10. Skin-friction Conduit Losses.* Many empirical equations for the uniform flow of water in conduits are based on the following expression, developed by Chézy in 1775:

$$v = CR^{0.5}S^{0.5}$$

or, in a more general form,

$$v = CR^{x}S^{y} \qquad [15]$$

where v = the average velocity in feet per second;
R = the hydraulic mean radius in feet, being the cross-sectional area of the water prism divided by the wetted perimeter;
S = sine of the slope of the energy gradient;
C, x, and y = coefficient and exponents, depending on the type of conduit and the condition of the wetted perimeter, all of which must be based upon experiments.

The results of most experiments on flow in open conduits have been published in terms of Kutter's equation.

Kutter's equation is

$$v = CR^{0.5}S^{0.5} \qquad [16]$$

where

$$C = \left[\frac{(1.811/n) + 41.66 + (0.00281/S)}{1 + [41.66 + (0.00281/S)]\, n/\sqrt{R}}\right] \qquad [16a]$$

Another equation for flow in open conduits has been devised by Manning. It is used where the approximate velocity as a function of the square root of the slope is desired and where a diagram for solving Kutter's equation is not available. For a range of n from about 0.012 to 0.020, the results are not materially different from those given by Kutter's equation.

There has been a recent tendency to use Manning's equation for pipes, but such practice is not recommended. Manning's is only a streamlined version of Kutter's equation, which is not adaptable to pipes.

Manning's equation is

$$v = CR^{2/3}S^{1/2} \qquad [17]$$

where

$$C = \frac{1.486}{n} \qquad [17a]$$

Recommended values for the friction coefficient, n, for use in Eqs. 16a and 17a are given in Table 3.

* Much of the text and many of the coefficients were compiled by Fred C. Scobey, member of the American Society of Civil Engineers. Editorial changes in formulas, to bring most of the equations to a common denominator in terms of the Williams-Hazen formula, required computation of coefficients.

TABLE 3

RECOMMENDED VALUES OF FRICTION COEFFICIENTS, n, FOR KUTTER'S EQ. 16 FOR OPEN CONDUITS

Dry Excavated Earth Canals:

Generally speaking, the value of n in earth canals increases with the life of the canal unless constantly maintained. Slightly silted waters will "slick" over an original rough surface so that the value of n becomes less. Heavily silted water will make n less, but it will also decrease the area of the water prism.

Best conditions are found in tough silt or clay soils, with velocities below scouring limits. $n = 0.016$ may be acquired by silt deposit free from growths.

New canals in sandy loam to clay loam range from the class above to the next one below.... .... $n = 0.020$

The accepted values for medium to large canals in firm earth or gravelly loam operated by organization that will give reasonable maintenance......... . . . . $n = 0.0225$

Small ditches, easily influenced by slight roughness, and larger canals poorly maintained . . . . . $n = 0.025$

Mountain power canals with cobble bottoms but without finer materials for a graded bedding ... . $n = 0.028$

Dredged Earth Canals:

A dredged channel is rougher than one excavated by hand or bulldozer. Likewise, a dipper dredge leaves a rougher bottom than does a drag line. Differences in value of n are largely brought about by adaptability of soil types and silt in the water to smooth over the original roughness.

Best conditions. .... $n = 0.0225$
Probable conditions. $n = 0.030$
Worst conditions without neglect of maintenance $n = 0.040$

Canals Excavated in Rock:

Net sections, neglecting possible overbreak, should be used.

It is possible for rock excavation to be done in horizontally stratified rock, resulting in a very smooth bottom. Such canals, if very wide, will have a low value of n, as the rough sides have relatively little influence and, if the canal is of ordinary size and no attempt is made to smooth up the sides materially, the minimum value of n is considered to be about.... . .. . $n = 0.023$

Usual or probable conditions, with care in smoothing the rock cut by breaking off projections . .. . $n = 0.033$

Worst possible conditions—there is no limit but it is seldom above $n = 0.040$

Silt and gravel deposits in rock canals may lower n by filling in the holes in the bottom.

TABLE 3—*Continued*

RECOMMENDED VALUES OF FRICTION COEFFICIENTS, n, FOR KUTTER'S EQ. 16 FOR OPEN CONDUITS

Natural Channels:

It is impossible to describe accurately the conditions of a natural channel that correspond to any given value of n.

The best natural channels have a value of n seldom below	$n = 0.025$
Average natural channels have a value of n probably in the neighborhood of	$n = 0.030$
Jungle bayous	$n > 0.10$
Worst possible conditions	No reasonable limit

Judgment and experience are necessary to fix even an approximate value of n for natural channels. As an aid in selecting a value of n, see Ref. 13 of Section 19, which gives some excellent photographs and descriptions of drainage channels for various values of n.

Concrete Linings:

The value of n depends on the specifications for the concrete surface and the workmanship. Considerable variation in n may result under the same specifications with different workmen. It should be borne in mind that more favorable values may be attained in construction than should be anticipated in design. The following are the general characteristics:

Best possible, with neat cement, extremely well-troweled surface. Seldom realized in practical construction	$n = 0.010$
The highest grade of practical concrete linings in best condition. Surface troweled as smooth as hand-troweled sidewalks. Expansion joints perfectly smooth.	
Best	$n = 0.011$
Probable	$n = 0.012$
Worst	$n = 0.013$
Surface as left by smooth-jointed forms, or roughly troweled. Expansion joints fair. The value usually adopted for concrete linings:	
Best	$n = 0.013$
Probable	$n = 0.014$
Worst	$n = 0.015$
Concrete having prominent form marks, or previous types subject to deposits of gravel on the bottom:	
Best	$n = 0.015$
Probable	$n = 0.016$
Worst or probable maximum value not subject to rejection because of bad workmanship	$n = 0.018$
If liable to a growth of moss, the foregoing values should be increased by adding	0.002

TABLE 3—*Continued*

RECOMMENDED VALUES OF FRICTION COEFFICIENTS, n, FOR KUTTER'S EQ. 16 FOR OPEN CONDUITS

Values of n in excess of 0.017 indicate very poor concrete, which may be either badly spalled, owing to frost or great difference in mixtures of concrete and finish coat, or broken down by very pure water or by the action of alkali. Similar values hold where the channel is losing its identity as a concrete surface, by deposits of sand and gravel or by moss or larval growths. Both moss and larvae appear to thrive in high velocities, even those of 30 or 40 ft per sec. Covering a channel to exclude sunlight is effective in preventing moss growths.

Gunite Linings:

Concrete linings deposited by a cement gun, from the inside:

Best, if lightly troweled.............................. $n = 0.014$
Good, if scrubbed with wire brush.............................. $n = 0.016$
Probable, if not scrubbed.............................. $n = 0.019$
Worst, for poor workmanship.............................. $n = 0.021$

Following a cement gun with a trowel will improve the surface from a capacity standpoint but may induce seepage loss by impairing the original density attained by the process. It is, however, recommended that the surface "rebound" be scrubbed off with a wire broom before it hardens and sticks to the canal bed.

Miscellaneous Masonry Linings:

Glazed brickwork:

Best.............................. $n = 0.011$
Probable.............................. $n = 0.013$
Worst.............................. $n = 0.015$

Brick in cement mortar:

Best.............................. $n = 0.012$
Probable.............................. $n = 0.015$
Worst.............................. $n = 0.017$

Dressed ashlar surface:

Best.............................. $n = 0.013$
Probable.............................. $n = 0.015$
Worst.............................. $n = 0.017$

For bench flume, consisting of natural rock surface for the uphill side, a smooth concrete retaining wall on the downhill side, and with a floor between. Floor lined with concrete and clean. Uphill side without projecting points.............................. $n = 0.020$

Same as above; but floor covered with sand or gravel, or left as excavated without projections, uphill side with a few projecting points, such as obtained with careful excavation in hard rock.............................. $n = 0.025$

TABLE 3—*Continued*

RECOMMENDED VALUES OF FRICTION COEFFICIENTS, n, FOR KUTTER'S EQ. 16 FOR OPEN CONDUITS

Cement—rubble surface:

Best	$n = 0.017$
Probable	$n = 0.025$
Worst	$n = 0.030$

Dry-rubble surface:

Best	$n = 0.025$
Probable	$n = 0.033$
Worst	$n = 0.035$

Wooden Box Flumes:

The following applies to well-constructed flumes, carefully maintained, without projecting calking or other imperfections. Battens where used to be included in wetted perimeter:

Planed lumber, longitudinal boards sides and bottom:

Best	$n = 0.011$
Probable	$n = 0.014$
Worst, after years of service	$n = 0.018$

Unsurfaced lumber, longitudinal boards sides and bottom:

Best	$n = 0.012$
Probable	$n = 0.015$
Worst, after years of service	$n = 0.018$

Roofing paper lining varies with the type, generally from . . . . $n = 0.010$ to $n = 0.017$

In general, the best and worst values correspond to new and very old flumes respectively, the latter with patches here and there in place of complete renewal of rotted members and corresponding to rather faulty work.

Wood-stave Flumes:

Creosoted:

Best	$n = 0.011$
Probable	$n = 0.012$
Worst	$n = 0.014$

Untreated:

Best	$n = 0.010$
Probable	$n = 0.012$
Worst	$n = 0.014$

Smooth-interior Steel Flumes:

For smooth-interior flumes, as manufactured and erected under various trade names:

TABLE 3—*Continued*

RECOMMENDED VALUES OF FRICTION COEFFICIENTS, n, FOR KUTTER'S EQ. 16 FOR OPEN CONDUITS

When unpainted:

Best	$n = 0.0105$
Probable	$n = 0.012$
Worst	$n = 0.014$

When painted:

Best	$n = 0.012$
Probable	$n = 0.013$
Worst	$n = 0.017$

The condition of these patent-joint flumes is largely a function of size and the number of carrying rods. Small flumes (2 to 5 ft in diameter) maintain their catenary shape quite well, if carrying rods are installed midway of each sheet, as well as at the ends, or if the side girders are set close to the sheets. Either method prevents excessive "scalloping." Large flumes require very frequent carrying rods and heavy-gage metal.

Williams and Hazen's equation for flow in closed channels was developed in 1902 from the meager data of that time to "represent as nearly as possible average conditions, as deduced from the best available records of experiments upon the flow of water in such pipes and channels as most frequently occur in waterworks practice" [7]. This equation is recommended for use with miscellaneous closed conduits, for which special equations will not be given.

Expressed in the form of Eq. 15, *Williams and Hazen's equation* is

$$v = 1.32C_M R^{0.63} S^{0.54} \qquad [18]$$

where C_M = the coefficient of friction.

Recommended values of the friction coefficient, C_M, for use in Eq. 18 are given in Table 4.

More recently, the writer, after years of field research, developed separate equations for steel, wood-stave, and concrete pipes.*

Given in a form similar to the preceding flow equations, these equations are as follows:

Scobey's equation for steel pipe is

$$v = C_s R^{0.58} S^{0.526} \qquad [19]$$

Scobey's equation for wood-stave pipe is

$$v = C_w R^{0.65} S^{0.555} \qquad [20]$$

Scobey's equation for concrete pipe is

$$v = C_c R^{0.625} S^{0.5} \qquad [21]$$

* For a detailed study of steel, wood-stave, or concrete pipe, see Refs. 8, 9, or 10, respectively, of Section 19.

Recommended values for the friction coefficients, C_s, C_w, and C_r, for use in Scobey's equations are given in Tables 5, 6, and 7, respectively.

Figure 11 gives values of R^x and S^y for use in the foregoing flow equations.

Figure 12 is for a solution of problems by Kutter's equation. Essentially the diagram consists of two parts: the upper part, which gives simultaneous values of R and n, and the lower part, which gives simultaneous values of S and v. The two parts are connected by a series of guide lines. The solution to a single problem is indicated by the dotted lines. If $R = 2.6$ ft and $n = 0.015$, then, for a slope of 0.00125, the indicated $v = 6.7$ ft per second. (The actual numerical answer is 6.66 ft per second.)

For slopes flatter than 0.0003 any given guide line splits into diverging curves. For the heavy guide lines the right-hand curves are lettered to indicate their use for values of $n = 0.030$ and the left-hand curves for $n = 0.012$. Of course, the right and left curves from the lighter guide lines are for the same respective values. Interpolation between proper guides can be estimated for other values of n.

In the selection of a proper coefficient for use in the aforementioned flow equations, the adopted value depends upon the judgment of the engineer. Since these equations embody different values of x and y, based on experiments for particular types of conduits, the value of C must be fitted according to the nature and condition of the wetted perimeter, the age of the conduit, and whether the maximum, probable, or minimum value of v is desired

The character of the wetted perimeter is liable to considerable change during the life of the conduit because of corrosion, tuberculation, growth of fungi, sponge, weeds and other vegetation, silt deposits, scour, ice, and other factors. The chief difficulties with existing experimental data are the lack of the power of definite description of the condition of the wetted perimeter of the conduit and the ever-present ignorance of the interior conditions of any conduit for any given year in the future.

For these reasons, it is necessary to adopt values of the friction coefficients that will result in an error, if any, on the side of safety. If, in a pipe line, the head lost in friction is only a small part of the total head available for power, it is customary when estimating power available to determine friction head by the selection of a probable coefficient. On the other hand, the design of a surge tank demands the selection of a coefficient corresponding to the minimum possible friction when the tank is filling, and the maximum possible friction when the tank is emptying. These selections result in the greatest possible surges in the tank.

For an open conduit, an error in the selection of the coefficient has considerable effect on the capacity. If a probable coefficient is used to determine the depth of water for a given discharge, a free board or superelevation of the sides of the conduit above calculated water surface is provided to insure a margin of safety and space for wave crests. This free board should be of such height that, with a coefficient corresponding to maximum possible

TABLE 4

RECOMMENDED VALUES OF FRICTION COEFFICIENTS, C_M, FOR WILLIAMS AND HAZEN'S EQ. 18 FOR MISCELLANEOUS CLOSED CONDUITS

Cast-iron Pipe:

On account of the growth of tubercles on the inside of the pipe, which decreases its area as well as increases its roughness, the value of C_M for a given age decreases as the diameter, but variation of C_M with age depends largely on the composition of the water flowing in the pipe Values are therefore rough approximations.

Williams and Hazen recommend the following average values of C_M:

Diameter of Pipe in Inches	Age in Years						
	0	5	10	20	30	40	50
4	130	118	107	89	75	64	55
8	130	119	109	93	83	73	65
12	130	120	111	96	86	77	70
16	130	120	112	98	87	80	72
24	130	120	113	100	89	81	74
30	130	120	113	100	90	83	76
36	130	120	113	100	90	83	76
40	130	120	113	100	90	83	77
60	130	120	113	100	90	83	77

Where cast-iron pipe is lined with cement it should classify with the best grade of concrete pipe, from a capacity viewpoint, if held to rigid specifications.

Small Smooth Pipe:

Williams and Hazen's coefficients for smooth brass, lead, tin, glass, drawn copper, and new, small steel pipe are given as follows:

New and in good condition	$C_M = 140$
Average .. .	$C_M = 130$
Worst.......	$C_M = 120$

Falling off of C_M with age depends on indeterminate conditions which are accentuated in small pipe. Therefore, an ample factor of safety is necessary.

Unlined Tunnels in Rock:

For unlined tunnels in rock, recommended values for C_M based on net sections and neglecting possible overbreak are as follows:

Best	$C_M = 50$
Probable ...	$C_M = 45$
Worst... . ..	$C_M = 38$

TABLE 5

RECOMMENDED VALUES OF FRICTION COEFFICIENTS, C_s, FOR SCOBEY'S EQ. 19 FOR STEEL PIPE

Scobey classifies steel pipe as follows:

Class 1. Full-riveted pipe, having both longitudinal and girth seams held by one or more lines of rivets with projecting heads. (Countersunk rivets belong in Class 3.)

1*a*. Sheet metal up to $\frac{3}{16}$ in. (about 7-gage) thick;
1*b*. Plate metal from $\frac{3}{16}$ in. to $\frac{7}{16}$ in. thick, with either taper or cylinder joints;
1*c*. Plate metal from ½ in. up with either taper or cylinder joints, and for plate from ¼ in. to $\frac{7}{16}$ in. thick when butt-jointed;
1*d*. Butt-strap pipe of plate from ½ in. up.

Class 2. Girth-riveted pipe, having no retarding rivet heads in the longitudinal seams, but with the same girth seams as full-riveted pipe.

Class 3. Continuous-interior pipe, having the interior surface unmarred by plate offsets or by projecting rivet heads in either longitudinal or girth seams. Not necessarily described as "smooth."

The following recommended conservative values are based on experience with eastern waters:

Class of Pipe	Age in Years					
	0	10	20	30	40	50
1*a*	140	130	120	111	103	95
1*b*	131	120	111	103	95	88
1*c*	125	115	106	98	90	84
1*d*	120	110	102	94	87	80
2	149	138	127	118	109	100
3	154	142	131	121	112	104

For western waters the above coefficients are multiplied by 1.0026^t, where t is the age in years.

TABLE 6

RECOMMENDED VALUES OF FRICTION COEFFICIENTS, C_w, FOR SCOBEY'S EQ. 20 FOR WOOD-STAVE PIPE

Best information seems to indicate that the value of C_w does not vary materially with age for wood-stave pipe. Experimental values differ, ranging as follows:

Best	$C_w = 224$
Probable	$C_w = 185$
Worst	$C_w = 170$

TABLE 7

RECOMMENDED VALUES OF FRICTION COEFFICIENTS, C_s, FOR SCOBEY'S EQ. 21 FOR CONCRETE PIPE AND CONCRETE-LINED TUNNELS

Concrete pipes are made in precast units in oiled, rigid forms, or they are centrifugally spun. Steel cylinders, with concrete interior and exterior, yield essentially the same surface.

Large tunnels, lined with concrete by means of a concrete gun and using oiled steel forms, will give a similar surface. Many experimental data on new precast pipe and concrete-lined pressure tunnels give the following values:

Best	$C_s = 145$
Probable	$C_s = 133$
Worst	$C_s = 121$

Wood formwork is little used in modern practice. Lack of rigidity causes the forms to squeeze under the pressure of wet concrete, resulting in offsets, crack fins, cavities, etc. Under such conditions $C_s = 130$ may be assumed for the best and $C_s = 90$ for the worst.

friction, the conduit will accommodate the depth required for the given discharge.

Many other cases can be cited where the minimum, maximum, or probable capacity of the conduit must be estimated.

The values in the following tables of coefficients were derived from a study of all available information on experimental determination of the friction coefficient and from the writer's numerous tests during 30 years with the Division of Irrigation in various bureaus of the U. S. Department of Agriculture [8–12]. The values corresponding to the best and worst conditions embrace practically all variations in the data studied. Isolated values, outside the limits given, have been excluded where there was reasonable doubt of their accuracy. It is felt that the values in the tables cover the range of practical considerations. The probable values are those ordinarily used in practice.

Unless otherwise stated the friction coefficients in the tables are based on straight or slightly sinuous conduits free from plant growths, moss, ice, etc. All experimental data were adjusted as closely as possible to correct for curvature so that all coefficients are for practically straight conduits. A correction to the calculated slope should be made for appreciable curvature, as indicated in Section 7.

Obviously, the results of defects in construction, settlement, and obstructions are indeterminate, although an attempt has been made in the tables to give approximate coefficients for certain conduits which should cover the range of reasonable maintenance.

Heavy coatings or growths of algae, fresh-water sponges, and the like are no respecters of material. Before the friction coefficient for a given case is adopted, similar conduits in the vicinity should be studied to determine

the possible effect of such coatings or growths on the coefficient. The possibility of treatment to control such organisms should also be investigated.

Effect of Ice. It is not known that any experiments have been made to determine the friction coefficient of the underside of a sheet of ice. If the ice had the same effect on friction as the bottom and sides of the conduit have, the determination of velocity would be made in the same way as for open-water conditions, except that the width of the ice sheet would be included in the wetted perimeter and the area would be the area under the ice. It is believed, however, that the usual retarding effect of the underside of an ice sheet is much less than that of the ordinary form of earth and rock canals and probably equal to that of the smoothest conduits. In the absence of a better method, it is recommended that, for ice conditions, Kutter's coefficient, n, for open-water conditions be used, and that a percentage of the width of the ice sheet, W, depending on the roughness of the conduit as indicated below, be included in the wetted perimeter.

$n = 0.010$	$W = 100$
0.015	87
0.020	76
0.025	66
0.030	58
0.035	50
0.040	43
0.045	37
0.050	32
0.055	28
0.060	25
0.065	22

The probable maximum thickness of the ice sheet depends upon climatic conditions at the site, the length, width, and depth of the conduit, and the velocity; and it can be estimated only by comparison with similar conduits in the vicinity.

11. Flow over Dams. (*a*) *Fundamental Formula.* The basic theoretical expression for the flow over weirs is given in Eq. 5 of Section 1, which, if all constants are combined, may be written:

$$Q = Clh_e^{3/2} \qquad [22]$$

where Q = the total discharge, in cubic feet per second;

C = the coefficient of discharge, which depends on the shape of the crest and the head on the crest;

l = the net or effective length of crest, i.e., the total length of crest corrected for end contractions due to piers and sharp-cornered abutments;

h_e = the actual or measured head on the crest, taken at a point sufficiently remote from the dam to avoid the surface curve.

Francis has determined that, to allow for the effect of the velocity of approach, this equation should be written:

$$Q = Cl[(h_c + h_v)^{3/2} - h_v^{3/2}] \qquad [23]$$

where h_v = the head corresponding to the velocity of approach.

An approximate form of Francis's equation is

$$Q = ql_n = Cl_n(h_c + h_v)^{3/2} \qquad [24]$$

For the same values of C, Eq. 24 gives values of Q in excess of those from Eq. 23, not exceeding about 1% for depths of channel of approach not less than twice the head on the crest.

(*b*) *End Contractions.* Francis's equation for the necessary correction due to complete end contractions is

$$l_n = l_t - 0.1n(h_c + h_v) \qquad [25]$$

where l_t = the total clear length of crest between abutments and piers and n = the number of complete contractions.

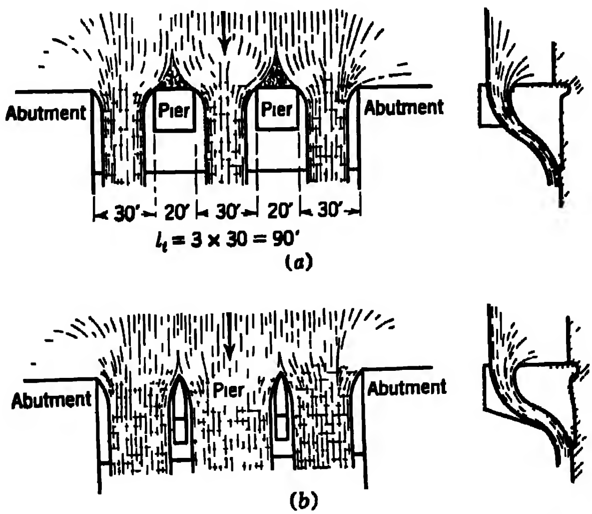


FIG 13 (*a*) Complete contractions due to abutments and large piers (*b*) Partial contractions due to sharp piers.

If the crest is obstructed by piers having considerable widths and sharp corners, as indicated in Fig. 13*a*, n represents the number of corners that serve to deflect the water. There are six complete contractions in this instance, two for each pier and one for each abutment. Usually, however, the piers are relatively thin and have rounded corners or sharp upstream ends, as indicated in Figs. 13*b* and 14. In such cases the contractions for the

piers are not complete, and Francis's equation would give too small values of l_n. Francis's equation then may be written

$$l_n = l_t - (h_e + h_v)(K_a n_a + K_b n_b \cdots K_n n_n) \tag{26}$$

where K_a, K_b, etc., represent the contraction coefficients applicable to the several different contractions that may be expected, and n_a, n_b, etc., the number of contractions having contraction coefficients, K_a, K_b, etc., respectively.

From a study of the data in the published experiments on the discharge capacity of the Wilson and Keokuk Dams [16, 17], as well as from other considerations, the authors have derived the approximate values of the coefficient of contraction, K, shown in Fig. 14, for piers having a thickness equal to about one third the head on the crest. Exact values can be found only by experimentation.

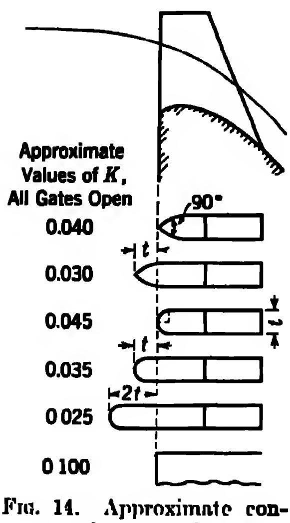


FIG. 14. Approximate contraction coefficients.

When one gate is open and the adjacent gates closed, each of the two end contractions may have a value of K equal roughly to 2.5 times that when all gates are open.

When the length of the weir between end contractions becomes short in relation to the head on the crest, the flow approaches that of discharge through an orifice and the principles of the weir no longer apply. In addition, the trajectory of the jet, fixing the shape of the crest of the dam, changes considerably. Therefore, when the length of crest between complete end contractions becomes less than about three times the head on the crest, or when the length of crest between piers of the usual type becomes less than about two times the head on the crest, the trajectory of the jet and discharge should be found by model experiments.

(c) *Velocity Head Correction.* The velocity head, h_v, introduced into Eqs. 23 and 24 to compensate for the motion of the water toward the weir, is found from the equation

$$h_v = \frac{V_a^2}{2g} \tag{27}$$

where V_a is the *effective* velocity of approach and g is the acceleration of gravity. The velocity in the channel of approach is not uniform. The filaments above the elevation of the crest sometimes have a velocity considerably greater than the mean, depending on the depth and width of the channel and its surface conditions. The energy of the filaments above the elevation of the

crest has a proportionately greater effect in increasing the discharge. The true value of V_a depends on too many variables to permit general determination. It may vary from 1.00 to 1.25 times the mean velocity.

Since wind, ice, and other conditions can materially affect the velocity of approach above the elevation of the crest, it is usual to assume the velocity of approach equal to the mean velocity. Such an assumption is on the safe side in determining the capacity of the dam to pass the maximum flood.

If the spillway is to serve as a measuring weir, it may be desirable to estimate the velocity of approach more carefully by study of experimental data for comparable installations, or by a model test.

If h_r is measured to a large pond, h_v may be neglected.

(d) Coefficients for Standard Dam Crests. The designer is interested in the coefficient of discharge for the following types of standard spillway dam crests:

Type 1. Straight dams with no velocity of approach.
- (a) Vertical upstream face.
- (b) Inclined upstream face.

Type 2. Straight dams with appreciable velocity of approach.
- (a) Vertical upstream face
- (b) Inclined upstream face.

Type 3. Curved dams with no velocity of approach.
- (a) Vertical upstream face.
- (b) Inclined upstream face.

Type 4. Curved dams with appreciable velocity of approach.
- (a) Vertical upstream face.
- (b) Inclined upstream face.

(e) Type 1a. Straight Dams—No Velocity of Approach—Vertical Upstream Face. Values of C for Eqs. 23 and 24, applicable to this type of crest, can be obtained from the "vertical water face" curve of Fig. 15. The "design head," h_c', in this figure is the head used in determining the shape of the crest. The first step in the determination of this curve is to find the coefficient of discharge C'' for the design head h_c' as follows:

It will be remembered that the method of construction of the standard dam crest corresponds to the shape of the jet from a theoretical sharp-crested weir, as shown in Fig. 16. Thus, for the design head, the discharge over the standard dam crest is the same as the discharge over the theoretical sharp-crested weir for the same elevation of water surface.

Francis's Eq. 23 was deduced from experiments on sharp-crested weirs, and the value of C was assumed constant at about 3.33 when the head was measured from the sharp crest, i.e., h_w in Fig. 16.

From Fig. 16 and also the table in Fig. 20 of Chapter 17, for a vertical face and zero velocity of approach

$$h_w = h_c' + r = 1.126h_c' \qquad [28]$$

where r is h_c' times the initial value of y in the table, and h_w is the depth of the theoretical sharp crest below the design water depth.

Consequently, to obtain the coefficient C'' applicable to the design head h_c' on the concrete crest, Francis's coefficient must be increased for use in Eqs 23 or 24 thus

$$C' = (1.126)^{3/2} C'_w = 3.33(1.126)^{3/2} = 3.98 \qquad [29a]$$

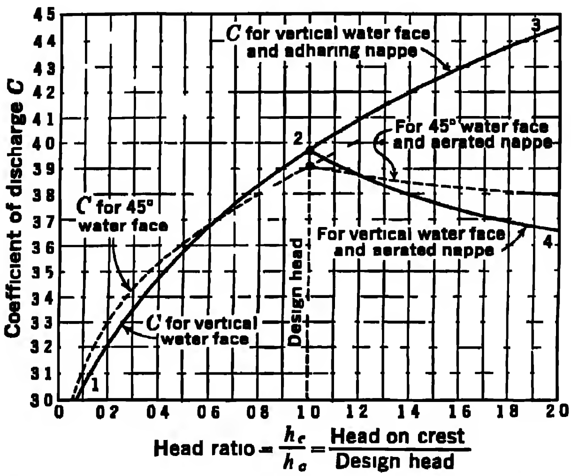


Fig 15 Coefficients of discharge for straight standard dam crests with no velocity of approach

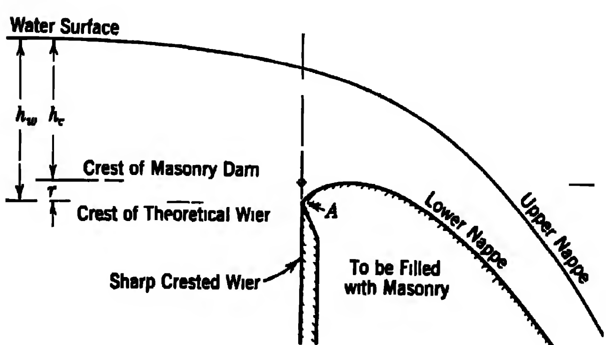


Fig 16 Nappe for sharp-crested weir with vertical upstream face

The value of 3.98 for C' (or of C Eqs 23 and 24) is good only for flow at the design depth In studying the performance of spillways it is necessary to estimate discharges for all depths and in such investigations C must be treated as a variable Discharge coefficients for standard crests for all ratios of actual head h_c to design head h_c', up to 2.0, can be taken from Fig 15

Point 2 on this diagram corresponds to a head ratio of 1.0; i.e., the actual discharge head is equal to the design head. The discharge coefficient is 3.98, as computed above. For actual heads smaller than the design head, the discharge coefficient is reduced, as indicated by the curve 1–2. For actual heads greater than the design head, there are two possibilities. If the overfalling stream adheres to the face of the dam, the value of C' increases as indicated by the curve 2–3. If the overfalling stream leaps clear of the dam and if the space between the dam and the lower nappe is fully aerated, there will be a decrease in the value of C' as indicated by the curve 2–4. It is only C' and not the unit discharge, $C'h_e^{3/2}$, that decreases with increasing values of h_e above the design head. Aeration is difficult if not impossible to obtain for standard dam crests.

Values of C' for heads less than the design head, as shown in Fig. 15, for a vertical-face dam, were deduced from a study of a number of experiments on many types of crests. The curve for values of C' greater than the design head was simply extrapolated. The curve for values of C greater than the design head, for the aerated nappe, was determined theoretically in the manner previously used for determining C'' for the design head.

Comparisons with the model experiments of Rouse and Reid [18] and those of Dillman in Munich [19] show that the curve agrees closely with experiment for values of h_e/h_e' from about 0.1 to 0.3 and that it indicates values of C' about 2% lower for values of h_e/h_e' of 1.0 (design head) and about 3% lower for values of h_e/h_e' of about 2.0. Thus the curves are comparatively conservative and within the accuracy obtainable in actual dams.

(*f*) *Type 1b. Straight Dams—No Velocity of Approach—Inclined Upstream Face.* For this type with a 45-degree upstream face, the coefficient of discharge, C'', for the design head in Fig. 15 was obtained in exactly the same manner as described for Type 1*a* (vertical upstream face), except that r was taken as 0.043 and C_w as 3.66, resulting in a coefficient, C'', of

$$C'' = (1.043)^{3/2}C_w = 3.66(1.043)^{3/2} = 3.91 \tag{29b}$$

There is evidence that when h_e/h_e' is less than about 0.6 the values of C' for a 45-degree water face are slightly greater than for a vertical face. No attempt was made to extrapolate the curve for a 45-degree face beyond the design head for the case of adhering jet, since no verification was available.

(*g*) *Type 2a. Straight Dams—Appreciable Velocity of Approach—Vertical Upstream Face.* The curves of Fig. 15 apply only to zero velocity of approach. As mentioned in Section 33 of Chapter 17, the shape of the standard crest changes with the velocity of approach. It is therefore evident that the coefficient of discharge also changes.

The coefficient of discharge for standard dam crests with an appreciable velocity of approach and designed to fit the jet is difficult to determine on account of lack of agreement of experiments. However, an approximate value of the discharge coefficient C'' may be obtained from

$$C' = 3.98 - \frac{0.48h_v}{h_c' + h_v} \qquad [29c]$$

where h_c' and h_v equal the "design head" and the velocity head, respectively.

(*h*) *Type 2b. Straight Dams—Appreciable Velocity of Approach—Inclined Upstream Face.* No data are available for this type. Experiments are necessary.

(*i*) *Type 3a. Curved Dams—No Velocity of Approach—Vertical Upstream Face.* The correct shape of the crest is indicated in Fig. 6, Chapter 26. Dams designed in accordance with this shape, with no velocity of approach, will have a coefficient of discharge as indicated in Fig. 17.

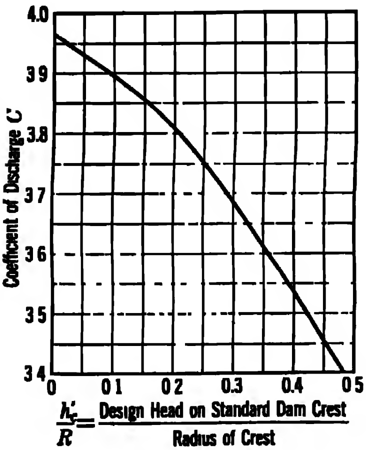


FIG. 17. Coefficients of discharge for curved standard dam crests with vertical upstream faces, operating at design head, no velocity of approach.

(*j*) *Types 3b and 4. Curved Dams.* No data are available for these types. Experiments are necessary.

(*k*) *Influence of Special Details.* For reasons explained in Section 33 of Chapter 17, the discharge coefficient is not appreciably altered by a re-entrance or lip as shown in Fig. 22*a* of the same chapter, or by a change in the 45-degree water face as shown in Fig. 22*b* of Chapter 17, provided that the distance *d* in both cases is equal to or greater than one half the sum of the head on the crest and the head corresponding to the velocity of approach.

(*l*) *Special Spillway Types.* Certain types of collapsible dams, intakes of chute spillways (see Chapter 26), and other wide flat intakes require special formulas.

It is not always possible to use the standard crest form for a spillway. Certain types of dam crests require crest forms of other kinds. Also, crests of many forms will be found on existing dams which the engineer is frequently called upon to analyze. In many cases no experiments have been made on such types of crests and approximations must be made from data obtained from similar types. Horton [15] has analyzed tests on a number of different types of dam crests, and these studies will be found very useful in such cases. However, the "broad-crested" weir, a type most frequently substituted for the standard crest, will be described later.

If Fig. 18*a* represents the upstream portion of a flat-top dam of appreciable width, or the intake to a chute spillway (Section 4, Chapter 26), or a similar conduit, the velocity at any point where the depth is *d* can be computed from the equation

$$V = \sqrt{2g(h_e + h_v - d - h_f)} \qquad [30]$$

where h_f is the total head loss due to contraction and friction between the pond level and the point at which d is measured. Other symbols are as shown in the figure.

The corresponding discharge is

$$Q = AV = ld\sqrt{2g(h_e + h_v - d - h_f)} \qquad [30a]$$

If the corner is well rounded to eliminate contraction and if the top of the dam is frictionless (or if friction is ignored), h_f is zero and Eq. 30 becomes

$$V = \sqrt{2g(h_e + h_v - d)} \qquad [31]$$

Eq. 30*a* likewise becomes

$$Q = AV = ld\sqrt{2g(h_e + h_v - d)} \qquad [31a]$$

If the top of the frictionless well-rounded dam is level, and if there is no obstruction to hold up the water depth on the crest artificially, the depth d will assume the minimum or critical value of

$$d = \tfrac{2}{3}(h_e + h_v) \qquad [32]$$

which may be substituted in Eq. 31*a* to give

$$Q = 3.087l(h_e + h_v)^{3/2} \qquad [33]$$

Of course, no dam top or spillway channel floor is frictionless; hence, if the floor is level, as indicated in Fig. 18*a*, for any considerable distance, the depth

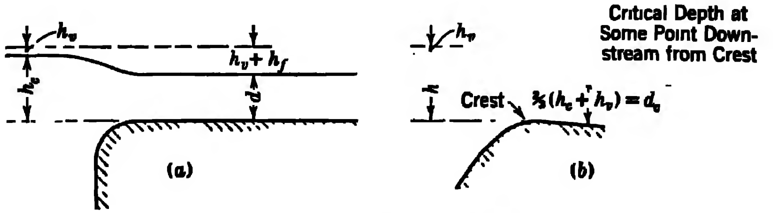


FIG. 18. Spillway intakes.

near the upstream edge of the crest will exceed the critical depth (Eq. 32). The usual procedure is to give the crest or channel floor a slope, as indicated in Fig. 18*b*, sufficient to balance the friction loss. The condition of frictionless flow is thus stimulated and Eq. 33 becomes applicable, provided the depth is not held up by some constriction further downstream. If the slope *immediately* below the crest is greater than required to compensate for losses, the discharge may be increased.

Equation 33 is generally used to compute the discharge of broad intakes for chute spillways. A small allowance for "entrance loss" can be made as a matter of safety. If the inlet is carefully designed, such loss will be small.

Equation 33 applies only to channel entrances or to structures where the upstream and downstream length is several times the head h_e.

Equation 30a is applicable to the discharge through restricted channels such as occur when cofferdams are built part way across a stream, and to temporary openings in dams to pass the stream during construction, as indicated in Fig. 19. If h_f is negligible, use Eq 31a If the depth of water below the restriction is less than $\frac{2}{3}(h_e + h_v)$, the value of d for use in Eq. 31a

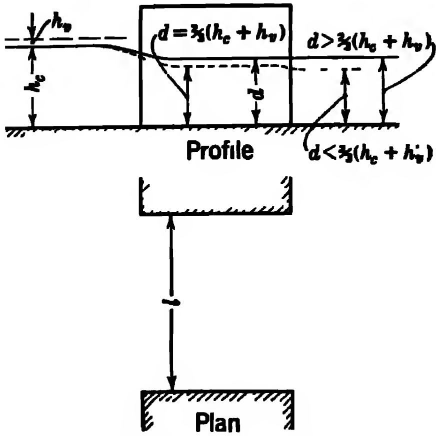


FIG 19. Flow through restricted openings.

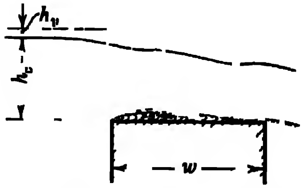


FIG. 20. Broad-crested weir not rounded

should be made equal to $\frac{2}{3}(h_e + h_v)$,* because the water surface through the restriction will be at that depth as previously explained. The value of l in Eq 33, as in all other cases, should be taken from Eq 25 or 26.

(*m*) *Broad-crested Weirs.* Flat-topped dams with square upstream corners (Fig. 20) are sometimes referred to as "broad-crested weirs" Discharge coefficients for such crests are irregular. If the head exceeds 1 5 to 2.0 times the crest width, the jet may jump clear of the crest, in which event the action is essentially that of a sharp-crested weir. Approximate coefficients for such weirs, based on available experimental data, are shown in Table 8 † ‡ These coefficients are for use in the approximate equation

$$Q = C_b l_n h_e^{3/2} \qquad [33a]$$

(*n*) *Submerged Spillways.* If the crest of the dam is submerged, as in Fig 21a, the discharge coefficient, for use in Eqs. 23 and 24, should be modi-

* Or Eq. 33 may be used, as it is the equivalent of Eq. 31a for $d = \frac{2}{3}(h_e + h_v)$

† Computed from Bazin's experiments in *U. S Geol. Survey Water Supply Paper* 200, by R. E Horton. Horton says these experiments apply to rising water surface only. Without aeration on top of the dam, the jet clears the crest at different heads for lowering water surface.

‡ Table is reproduced by permission from Horace King, *Handbook of Hydraulics*, McGraw-Hill Book Co., New York, 1939, p 164.

TABLE 8

VALUES OF C_b IN THE FORMULA $Q = C_b l_n h_c^{3/2}$ FOR BROAD-CRESTED WEIRS

[Applicable to rising water surface only]

Measured Head in Feet, h_c	Breadth of Crest of Weir, feet										
	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

fied according to the degree of submergence, as indicated in Table 9.* In this table, C is the coefficient for *free* discharge over a similar crest under the same head, and C_s is the modified coefficient due to the submergence. The heads are h_s and h_c, as in Fig. 21*a*.

It will be noted that, for values of h_s/h_c less than 0.30, the reduction in discharge is less than 3%.

If a standing wave occurs below the crest, as indicated in Fig. 21*b*, the effect of submergence is lost and the discharge is the same as for a free crest.

(*o*) *Discharge through Partly Open Crest Gates.* It is not possible within the scope of this book to give data on the discharge through partly open crest

* From U. S. Deep Waterways experiments. See *U. S. Geol. Survey Water Supply Paper* 200, p. 146. These experiments were made on a model having a rounded crest, approximating more closely than any of the others to the shape of a standard dam crest.

TABLE 9

RELATIVE COEFFICIENTS, SUBMERGED CREST AND FREE CREST

$\frac{h_s}{h_c}$	$\frac{C_s}{C}$	$\frac{h_s}{h_c}$	$\frac{C_s}{C}$
0.0	1.000	0.6	0.907
0.1	0.991	0.7	0.856
0.2	0.983	0.8	0.778
0.3	0.972	0.9	0.621
0.4	0.956	1.0	0.000
0.5	0.937		

gates, since the varied details of gates and supporting structures affect conditions materially. The reader is referred to the following publications:

Experiments on Wilson and Keokuk Dams, Refs. 16 and 17.
Robert E. Horton, "Discharge Coefficients for Taintor Gates," *Eng. News-Record,* Jan. 4, 1934, p. 10.
Theron M. Ripley, "Discharge through Taintor Gate Openings," *Civil Eng.,* August 1933, p. 386; Ben Gumensky, idem, November 1933, p. 627.
Julian Hinds, "Rating Curves for Canal Headgates," *Reclamation Record,* May 1922.
Julian Hinds, "Discharge Coefficients for Canal Headgates," *Reclamation Record,* October 1919.

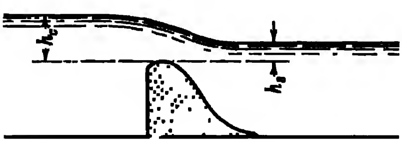


FIG. 21a. Submerged crest.

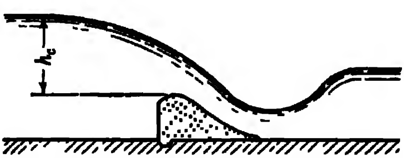


FIG. 21b. Submerged crest with standing wave.

12. Measuring Weirs. Many formulas for determining the flow over measuring weirs have been proposed,* each giving slightly different results. The use of such formulas should be confined to conditions exactly equivalent to those in the experiments from which the formulas were derived.

The August 1942 supplement to the Power Test Code (1938) of the American Society of Mechanical Engineers for hydraulic prime movers specifies the use of Rehbock's formula as follows:

$$Q = \left(3.228 + 0.435\frac{h_e}{z}\right) l h_e^{3/2} \qquad [34]$$

where Q = quantity in cubic feet per second;
$h_e = h + 0.0036$;
h = observed head on crest, in feet, without correction for velocity of approach;
z = height of weir crest above bottom of channel of approach, in feet;
l = length of weir crest, in feet.

* See p. 86, Ref. 1 of Section 19.

This code also provides a specification for measuring hydraulic turbine discharge It will apply, of course, to measurements of streams.

13. Head by Bernoulli's Theorem. The velocity head, or head required to produce a given velocity, was derived in Section 4 and is

$$\text{Velocity head} = h_v = \frac{v^2}{2g} \qquad [35]$$

According to Bernoulli's Theorem, the general law governing steady flow of water in conduits is as follows:

For steady flow in a conduit, the sum of the velocity head, the pressure head, and the potential head at any point, *A*, is equal to the sum of the corresponding heads at any upstream point, *B*, less the frictional resistance between the points *A* and *B*

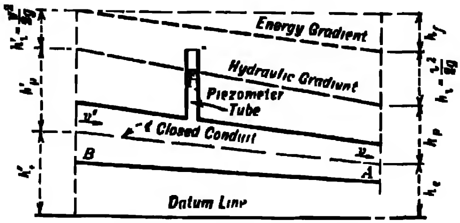


Fig. 22. Bernoulli's Theorem.

Expressed mathematically, we have

$$\frac{v^2}{2g} + h_p + h_e = \frac{v'^2}{2g} + h_p' + h_e' - h_f \qquad [36]$$

where v and v' = the velocities in feet per second at points *A* and *B*, respectively;
h_p and h_p' = the corresponding pressure heads, in feet;
h_e and h_e' = the corresponding potential heads above a common datum plane, in feet;
h_f = the total frictional resistance or lost head, in feet, between points *A* and *B*.

In a closed conduit under pressure, h_p is the height at which water will stand in a piezometer tube, as indicated in Fig. 22, and defines the elevation of the hydraulic gradient above the conduit; and $h_p + v^2/2g$ defines the corresponding elevation of the energy gradient.

In an open conduit, h_e and h_e' are measured to water surface and h_p and h_p', measured to the same place, are equal to zero. Therefore, the hydraulic gradient for open conduits is at water surface.

For water-power developments, an additional head, h, must be included in the theorem. This head is that used by the turbines in the generation of power and includes all frictional resistance within the turbine casing, the turbine, and the draft tube, since such losses are chargeable against the efficiency

of the turbine. The special case of Bernoulli's Theorem, applicable to water power, is

$$\frac{v^2}{2g} + h_p + h_e = \frac{v'^2}{2g} + h_p' + h_e' - h_f - h \qquad [37]$$

For incased turbines, let the points A and B be located at the upper end of the tailrace and in the end of the penstock at the turbine casing, respectively. In the tailrace, $h_p = 0$ as it is an open conduit.

The only loss between A and B, not chargeable against turbine efficiency, is that due to the sudden enlargement at the end of the draft tube. If v_d is the velocity at the end of the draft tube, the loss may be assumed to be the difference in the velocity heads in the draft tube and in the tailrace, or

$$h_f = \frac{v_d^2}{2g} - \frac{v^2}{2g} \qquad [38]$$

Making the proper substitutions in Eq. 37, we have

$$h = h_p' + (h_e' - h_e) + \frac{v'^2}{2g} - \frac{v_d^2}{2g} \qquad [39]$$

Equation 39 agrees with the definition of the A.S.M.E. Power Test Code (1938) for the net head acting on incased reaction turbines.

For reaction turbines in open flumes, h_p' is zero and $v'^2/2g$ is negligible. Therefore, for such installations, the equation is

$$h = h_e' - h_e - \frac{v_d^2}{2g} \qquad [40]$$

Equation 40 corresponds to the definition of the A.S.M.E. Power Test Code (1938), for the net head acting on turbines in open flumes.

For impulse turbines, let the point A be located at the jet where it becomes tangent to the bucket circle, and point B in the penstock at the nozzle casing. After leaving the runner,* $h_p = 0$ and $h_v = 0$. The only loss between A and B is that in the nozzle, which is chargeable against turbine efficiency; therefore, $h_f = 0$. Consequently, Eq. 37 reduces to

$$h = h_p' + (h_e' - h_e) + \frac{v'^2}{2g} \qquad [41]$$

This equation corresponds to the definition given in the A.S.M.E. Power Test Code (1938) for the net head acting on impulse turbines.

* Theoretically $h_v = 0$. Actually the water has velocity in a vertical direction necessary to drop to the tailrace level the energy of which is lost, and sometimes it has a horizontal velocity due to imperfections in design, the energy of which is chargeable against turbine efficiency.

For the whole development, let A and B be located in the river where the water is returned and where it is diverted, respectively. The pressure head at these points is zero; the velocity at the point of diversion is usually negligible; and that at the point of return is either zero or not available for power. Therefore,

$$\frac{v^2}{2g} = \frac{v'^2}{2g} = h_p = h'_p = 0$$

and Eq. 37 reduces to

$$h = h_e' - h_e - h_f \qquad [42]$$

From the definition of gross head given in the A.S.M.E. Power Test Code (1938),

$$H = h_e' - h_e$$

Therefore,

$$h = H - h_f \qquad [43]$$

The net head on the turbine is therefore equal to the gross head, or the difference in elevation between the water surface in the diversion pond and that in the river at the lower end of the tailrace, less all frictional losses in the conduits not chargeable against turbine efficiency.

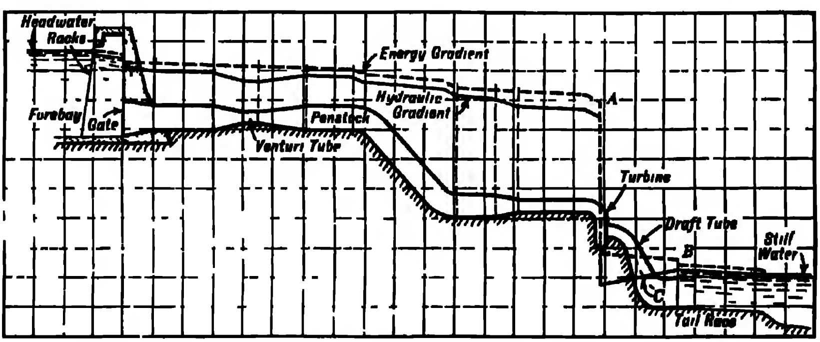


FIG. 23. Hydraulic and energy gradients for a complete development.

Figure 23 shows the hydraulic and energy gradients for a complete development. The hydraulic gradient is at all places a vertical distance below the energy gradient equal to the velocity head at such places. The energy gradient at any point is a vertical distance below headwater elevation equal to the lost head (total friction losses) between the forebay and that point. Therefore, the hydraulic gradient at any point is a vertical distance below headwater elevation equal to the total friction losses between the forebay and that point plus the velocity head at that point.

The net head available for power is the vertical distance between points A and B, since, as previously explained, the friction losses in the turbine and draft tube are chargeable against turbine efficiency.

It will be noted that the pressure in the draft tube above point C is negative as the hydraulic gradient is below the tube.

The conduit above the turbine should be well below the hydraulic gradient to avoid negative pressures.*

14. Critical Gradients and the Hydraulic Jump. Figure 24 is a longitudinal section of an open conduit in which the water is flowing at a depth d

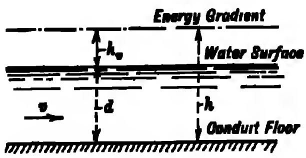


FIG. 24. Longitudinal section of open conduit.

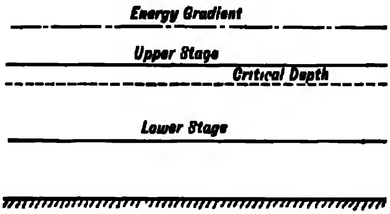


FIG. 25. Alternate depths in an open conduit.

and at a velocity v. As indicated in Section 13, the energy gradient lies at an elevation above the water surface equal to the velocity head, or $h_v = v^2/2g$. From Fig. 24,

$$h = d + \frac{v^2}{2g}$$

But, if Q is the discharge per foot of width,

$$v^2 = \frac{Q^2}{d^2}$$

Combining, we have

$$h = d + \frac{Q^2}{2gd^2}$$

which may be reduced to

$$Q = 8.02\sqrt{d^2h - d^3} \qquad [44]$$

Equation 44 indicates that for given values of Q and h there are three possible values of d, one of which, however, is imaginary. Therefore, for a given discharge, Q, and elevation, h, of the energy gradient, there are two depths, d, at which the water will flow, provided the slope of the conduit is suitable for the corresponding velocity and hydraulic radius. This is indicated in Fig. 25. If the energy gradient remains fixed and the discharge increases, the two stages approach each other and are coincident at the critical depth when the conduit is discharging the maximum flow of which it is capable.

* So far, only steady flow of water has been considered. For unsteady flow due to water hammer, the hydraulic gradient will be much lower than here described and allowance must be made for this lowering in fixing the elevation of the conduit.

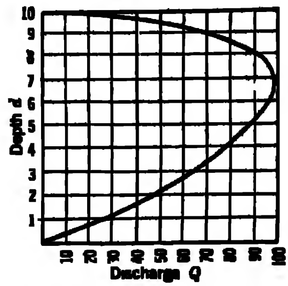


FIG. 26. Example of alternate depths in an open conduit.

As a practical example, let $h = 10.0$ ft. The curve of Fig. 26 has been plotted by substituting this value of h, together with various values of d, in Eq. 44, the width of conduit used being unity. This curve clearly shows the alternate depths for each discharge of which the conduit is capable with a constant value of $h = 10.0$ ft. It also shows that the maximum possible discharge is 97.7 sec-ft, corresponding to a depth, d, of $2h/3$ or 6.666 ft where the two stages are coincident.

That the maximum discharge occurs for $d = 2h/3$ can be proved by making the first differential of Q with respect to d in Eq. 44 equal to zero; whence,

$$\frac{(dQ)}{(dd)} = \frac{\overline{8.02^2}}{2Q}(2dh - 3d^2) = 0$$

from which

$$d = \frac{2h}{3} = \frac{v^2}{g} = 2h_v \qquad [45]$$

The depth $d = 2h/3$ is called the "critical depth."

The slope of a channel which will maintain a given discharge at the critical depth is called the "critical slope." For a slope flatter than the critical slope,

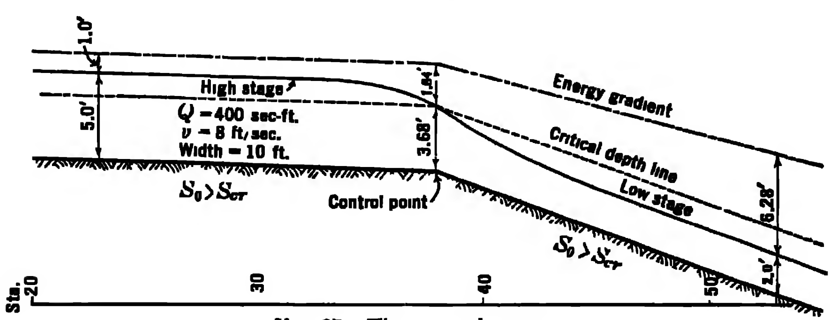


FIG. 27. The control point.

as at the left of Fig. 27, the flow will be at high stage. If the slope is greater than the critical slope, as at the right of Fig. 27, the flow will be at low stage. The point where the flow changes from the high stage to the low stage, or where the depth is equal to the critical depth, is called the "control point."

Uniform depths near the critical depth should be avoided in open-conduit design, for, under such conditions, very slight irregularities in the conduit section may cause the water to fluctuate between the two stages with great disturbance and increased friction.

Flow conditions occur where the water surface changes from a low stage to a high stage. For example, a jet from an orifice as in Fig. 29*b* and the discharge over a spillway as in Fig. 30*a* contain flow at low stage jumping to the high stage, if the depth of tailwater is great enough. The phenomenon of this abrupt increase in depth of flow is termed the "hydraulic jump."

The equation for the hydraulic jump in horizontal channels without entrained air, as given by Gibson [20] and others, is

$$D = \frac{d}{2}\left(\sqrt{\frac{8v^2}{gd} + 1} - 1\right) \qquad [46a]$$

where D = the depth of the high stage in feet;
d = the depth of the low stage in feet;
g = the acceleration of gravity = 32.2 ft/sec²;
v = the velocity at the low stage in feet per second.

Where entrained air is present as in the flow in steep chutes (described in Section 16), the equation for the hydraulic jump, as given by Douma,* is

$$D^2 = \rho d^2 + \frac{2\rho d v^2}{g}\left(1 - \rho\frac{d}{D}\right) \qquad [46b]$$

where ρ = the ratio of the volume of water to the total volume of air and water at a given cross-section;
d = the actual low-stage depth of air and water;
D = the actual high-stage depth, assuming that no entrained air remains after the jump.

For the hydraulic jump on slopes see Ref. 21 of Section 19. However, Eqs. 46*a* and 46*b* can be applied to slopes flatter than 1 on 4. In steeper

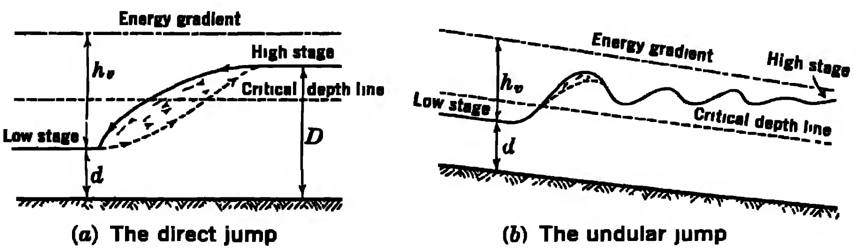


FIG. 28. Two types of hydraulic jump.

slopes the jump is likely to be drowned out; the jet penetrates tailwater, and high velocities continue for a considerable distance downstream.

The hydraulic jump as given by Eqs. 46*a* and 46*b* may be a direct jump, as shown in Fig. 28*a* or it may be an undular jump as shown in Fig. 28*b*.

If h_v is greater than 1.5 d or $D > 2d$, as is usual, the jump will be direct. There will be a rapid dissipation of energy and a gradual reduction of velocity

* See p. 1473 of Ref. 27 of Section 19.

from the beginning to the end of the jump. This is the best form of jump because the flow at the lower end is well distributed throughout the depth of water.

If, however, for very low dams with large discharge, h_v is less than $1.5d$ or $D < 2d$, Bakhmeteff [23] has shown that the jump will be undular and does not have such rapid energy dissipation, but involves a train of standing waves extending some distance downstream. Section 5 of Chapter 25 discusses bottom protection for the undular jump.

The hydraulic jump can occur only if the flow is at low stage, i.e., if the water surface is below the "critical stage." From Eq. 45, the flow is at low stage if

$$d = < 2h_v = < 2\left(\frac{v^2}{2g}\right).$$

That is,

$$d = < \frac{v^2}{g} = < d_{cr}$$

Thus, the hydraulic jump is possible only if

$$v = < \sqrt{gd} \qquad [47]$$

Figure 29*a* represents a flume 10 ft wide designed to carry 400 sec-ft with normal water surface in the forebay and with the entrance gate wide open. It requires a depth of 5 ft and a velocity of 8 ft per second. The water flows at high stage.

Figure 29*b* represents the conditions of flow during high forebay level when the gate is only partly open to limit the flow to the required 400 sec-ft. The water issues from the gate at low stage, and, if the flume had sufficient slope to overcome friction at this high velocity, the water would continue to flow at this depth. However, the flume has a slope sufficient only for the low velocity and large hydraulic radius of the high-stage flow. Therefore, the friction loss is greater than for the slope of the flume, and the depth of water increases. The jump will occur when the depth at low stage has increased to 2.6 ft; this, from Eq. 46*a*, is the depth required to produce the jump for a 5-ft depth in the flume.

Figure 30*a* shows the hydraulic jump at the toe of a spillway dam. The water passes from the high stage to the low stage at the crest and flows down the face of the dam and into the lower river at low stage. The depth, *d*, is difficult to compute accurately. Empirical methods have been proposed.* For safe approximations the depth, *d*, can be assumed equal to the thickness of the free-falling jet indicated in Figs. 20 and 21 of Chapter 17.

The normal high-stage elevation of the water surface in the lower river, corresponding to the discharge over the dam, should be determined from a rating

* See p. 75 of Ref. 14 of Section 19.

curve of the river. If the corresponding depth D of the tailwater is greater than that indicated by Eq. 46, the jump will not occur on the apron but on the bucket of the dam. If the depth is less than indicated by Eq. 46, the jump will occur in the river bed downstream from the apron. The river bed will then be subjected to the high velocities of the low-stage flow, resulting in erosion.

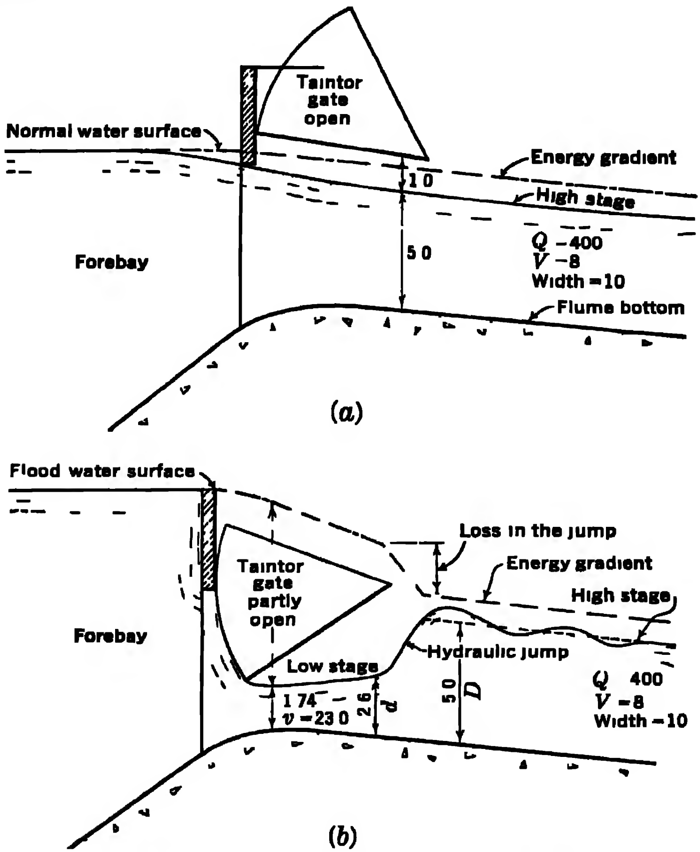


FIG. 29. Creation of hydraulic jump under gate.

For a jump occurring downstream from the apron, one method of obtaining the required depth and causing the jump to occur on the apron is to lower the apron, as shown in Fig. 30b, creating what is known as a "stilling basin." For other methods of erosion control, the reader is referred to Art. 29, Chapter 3, of Ref. 14, Section 19.

In determining the elevation of the water surface in the lower river, care must be taken to see that it is conservatively low, since a lower elevation than the one assumed would throw the jump off the apron.

Bakhmeteff and Matzke* indicate that the length of the hydraulic jump in horizontal channels—that is, the horizontal distance required for the reduction of bottom velocity to mean tailwater velocity—will not exceed about 5*D*. Blaisdell [24] has shown that the length of the hydraulic jump can be mate-

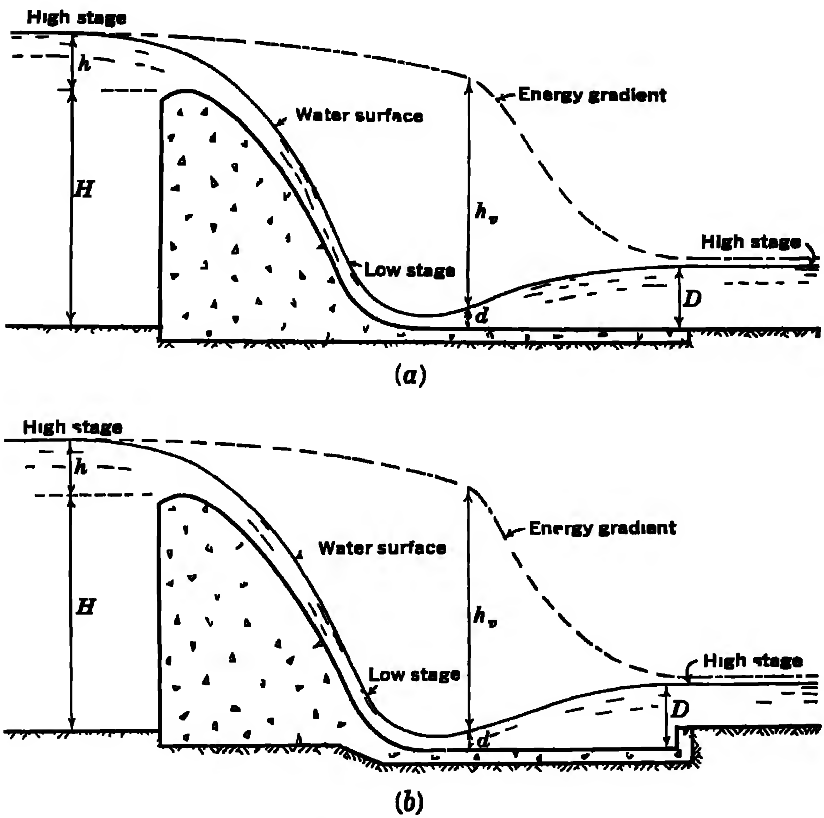


Fig. 30 Examples of hydraulic jump

rially reduced with a proper arrangement of chute blocks, baffles, and end sills.

15 The Hydraulic Bore and the Suction Wave in Open Channels. Figure 31 is a profile of a typical open conduit. The normal depth at constant flow per unit width of conduit q is d and the corresponding velocity is v. If the flow out of the lower end is suddenly stopped by a closure of the turbine gates, the water surface at the lower end will immediately rise and the hydraulic bore, or pressure wave, AB will advance up the conduit with a velocity of propagation, a. The discharge, q, rejected by the turbines will

* See p. 643 of Ref. 22 of Section 19

fill the space $ABCE$ and create automatically an excess head or force to decelerate the flow in the conduit progressively as the bore advances, the velocity below the bore being zero and that above the bore being v.

A knowledge of the height, AB, of the bore is necessary in order to fix the height or superelevation of the sides of the conduit.

The hydraulic bore is simply another form of the standing wave, described in Section 14. To apply Eq. 46a for the standing wave to the case of the

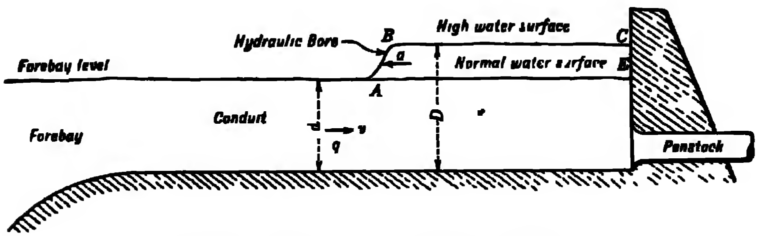


FIG. 31. Hydraulic bore without friction.

hydraulic bore, it must be remembered that the velocity of flow, v, is the velocity past the standing wave, and that, for the hydraulic bore advancing with a velocity a, the velocity of flow past the wave is $v + a$. Therefore, an equation for the hydraulic bore can be obtained by substituting $v + a$ for v in Eq. 46a.

But

$$a = \frac{q}{D - d} = \frac{dv}{D - d}$$

Therefore,

$$v + a = \frac{vD}{D - d}$$

And, making this substitution for v in Eq. 46a, we have

$$D = \sqrt{\frac{2d}{g}\left(\frac{vD}{D - d}\right)^2 + \frac{d^2}{4}} - \frac{d}{2} \qquad [48]$$

which is the equation of the hydraulic bore.

This equation can be solved only by trial. The value of d is entered in the equation, and then different values of D are adopted and substituted until the second term of the equation equals the value of D adopted. To assist in the solution of this equation, an approximate value of D may be obtained from the following empirical equation by Kennison [25], which is in more convenient form than Eq. 48:

$$D = \frac{v\sqrt{d}}{5} + 0.90d \qquad [49]$$

So far, friction in the conduit has not been considered. Figure 32 shows a practical case in which a slope occurs in the normal water surface. In such circumstances the discharge rejected by the turbines must also fill the volume BCF, which grows larger as the bore advances toward the forebay. This causes a reduction in the velocity, a, and hence a reduction in depth, D, because the residual velocity between the bore and the lower end of the conduit is not zero but has a value equal to that required to fill the ever-increasing volume in the triangle BCF. However, this residual velocity must be eliminated after the bore reaches the forebay, and thus an additional rise of water surface will be required. Thus, there is a tendency in practice towards both a reduction and an increase in the value D, but both are relatively small and can be considered to counteract each other.

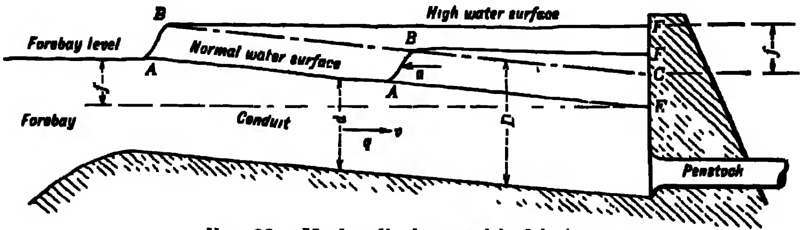


FIG. 32. Hydraulic bore with friction.

Therefore, for the practical case, the maximum depth of water in the conduit can be considered equal to D from Eq. 49 at the forebay and, at the lower end of the conduit, equal to D plus the original conduit friction loss, f (Fig. 32). The high-water surface would then be at the same level throughout the conduit.

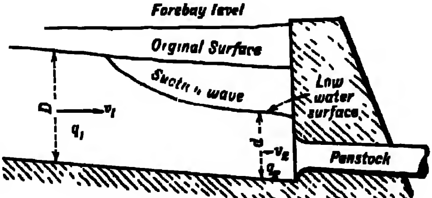


FIG. 33. Suction wave.

A sudden increase in the demand for water in the lower end of an open conduit will result in a sudden drop in the water level at that end. The lower level and the resulting increase in slope serve to accelerate the water to meet the increased demand. This phenomenon, indicated in Fig. 33, is known as the suction wave and is the direct opposite of the hydraulic bore, except that it does not travel up the conduit as a constant wave; instead, the slope of the wave becomes flatter as it moves towards the upper end. In this example, practical interest is at the lower end of the conduit, where

the minimum depth, d, must be known for expected load changes. R. D. Johnson [26] gives as the equation of the suction wave:

$$D - d = (v_2 - v_1)\sqrt{\frac{d}{g}} + \frac{(v_2 - v_1)^2}{4g}$$

But

$$v_2 = \frac{q_2}{d}$$

Therefore,

$$D - d = \left(\frac{q_2}{d} - v_1\right)\sqrt{\frac{d}{g}} + \frac{\left(\frac{q_2}{d} - v_1\right)^2}{4g} \qquad [50]$$

16. Varied Flow. (*a*) *General.* The flow in open channels having a variable cross-section or slope is known as varied flow. Problems in varied

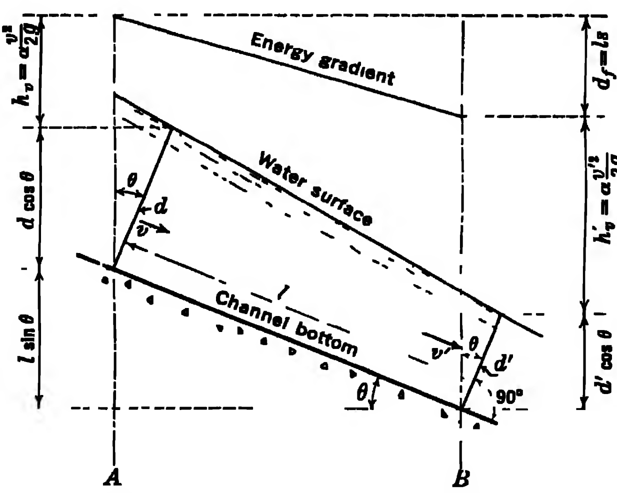


FIG. 34 Application of Bernoulli's Theorem for open conduits

flow can be approached most readily by Bernoulli's Theorem, which, as indicated in Fig 34, can be written thus for open channels between two stations,

$$l \sin\theta + d\cos\theta + \frac{\alpha v^2}{2g} = d'\cos\theta + \frac{\alpha v'^2}{2g} + lS \qquad [51]$$

or

$$d' = \frac{l\sin\theta + d\cos\theta + (\alpha/2g)(v^2 - v'^2) - lS}{\cos\theta} \qquad [51a]$$

where l = the length along the bottom of the channel between any two stations, in feet;

d and d' = the depth of water, in feet, measured normal to the channel bottom;

v and v' = the velocity, in feet per second, parallel to the channel bottom;

θ and θ' = the angle of slope of channel bottom with the horizontal;

α = the mean velocity coefficient, which, according to Hall [27, 28], normally ranges from about 1.05 for smooth channels to 1.09 for fairly rough channels;

g = the acceleration of gravity = 32.2.

(*b*) *Steep Chutes* Water moving in steep chutes is one form of varied flow. This type of flow entrains air and causes an increase in the area of the section.

The following procedure, prepared by L. Standish Hall,* for the solution of steep chute problems involves the cut-and-try method:

For a channel without air entrainment, corresponding to the two stations in part (*a*), let

Q = the discharge, in second-feet;

b = the width of the water surface, in feet;

b_1 = the bottom width of the channel, in feet;

s = the cotangent of the angle of the side slopes with the horizontal, in trapezoidal sections;

A and A' = the cross-sectional area of the channel, in square feet;

R and R' = the hydraulic radius, in feet;

n = Kutter's coefficient of friction (from Table 3);

K = the air-entrainment coefficient (from Table 10).

For actual conditions with air entrainment, the subscript a is used

Step 1. Assume a value of n from Table 3 for the air-water mixture equal to the value normally used without air entrainment for the appropriate surface, and a value of K from Table 10.

TABLE 10

RECOMMENDED VALUES OF THE AIR-ENTRAINMENT COEFFICIENT †

Type of Surface	Values of K
Plank flumes without battens	0.003 to 0.004
Neat cement or smooth metal	0.003 to 0.004
Cement mortar or average concrete	0.004 to 0.006
Rough concrete or smooth-dressed ashlar	0.008 to 0.012
Rough ashlar or smooth cement-rubble	0.015 to 0.020

† *Op. cit.*

* Formerly Principal Hydraulic Engineer, East Bay Municipal Utility District, Oakland, Calif.

Step 2. Since, for all steep chutes, the water passes from the upper gradient (see Section 14) to the lower gradient at the control, which is usually at the summit of the channel, find the location of the control and the values of d, A, R, and v on the assumption of no air entrainment.

For rectangular channels

$$= \sqrt[3]{\frac{\alpha Q^2}{b^2 g}} \qquad [52a]$$

For trapezoidal channels

$$Q\sqrt{\frac{\alpha}{g}} = A\sqrt{\frac{A}{b}} \qquad [52b]$$

Step 3. At this point of control, compute the value of ρ, the ratio of the volume of water to the total volume of air and water, from

$$\rho = \frac{1}{1 + (Kv^2/gR)} \qquad [53]$$

Step 4. Determine the actual values of A_a, R_a, and d_a, with air entrainment, all based on the value of

$$\rho \qquad [54]$$

For rectangular sections

$$\rho$$

For trapezoidal sections

$$d_a = \frac{-b_1 + \sqrt{b_1^2 + (4sQ/\rho v)}}{2s} \qquad [55b]$$

Step 5. Also, for this point of control compute n_e, the effective roughness coefficient, from the equation

$$n_e = n\left(\frac{R}{R_a}\right)^{2/3} \qquad [56]$$

Step 6. Divide the channel into a number of reaches of length l, using small values of l when the velocity accelerates rapidly and larger values when the flow becomes more uniform. The first reach will start at the point of control.

For each reach, values of A, R, v, ρ, N_e, A_a and R_a are known at the beginning or upper end, as at Sta. A of Fig. 34.

Step 7. For the lower end of the reach, shown as Sta. B in Fig. 34, assume a tentative value of d' and calculate R', v' and n_e'. If desired, subsequent calculations are facilitated by preparing a series of curves of the several functions of each section for varying depths.

Step 8. Average the values of R and R', v and v', and n_e and n_e' between the two sections A and B.

Step 9. With these average factors, find the average sine of the slope of the energy gradient, S, and the loss, lS, of energy between sections A and B from Equation 16a, using Fig. 12.

Step 10. From Eq. 51a find the corresponding value of d', which should agree with the tentative value assumed in Step 7. If it does not agree, assume another tentative value of d' and repeat the steps.

In this way, successive reaches are calculated to the end of the channel or until the energy gradient becomes parallel to the bottom of the channel.

Step 11. After each reach is computed, find d_a', and this will locate the final water surface with air entrainment.

If the chute has a uniform cross-section, either rectangular or trapezoidal, and a uniform slope, a direct method of computation is available * eliminating the "trial-and-error" method.

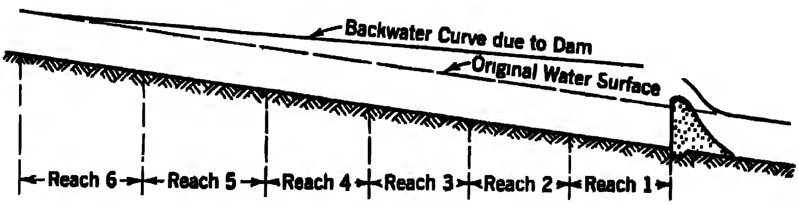


FIG. 35. Backwater curve.

Care must be taken to prevent the water from jumping clear at places where there is a sudden increase in slope. The profile of the bottom of the channel should be well within the parabola that forms the trajectory of the jet at the point of grade change with the trajectory computed with a velocity of 1.25 times the mean velocity in the cross-section.

For details of design at horizontal curves see Refs. 29, 30, and 31 of Section 19. For the dissipation of the energy at the foot of the chute or for other means of protecting the foundation, see Section 5 of Chapter 25.

(*c*) *Backwater Curves*. The backwater curve, upstream from a point of control such as a dam, as indicated in Fig. 35, is another example of varied flow. In the case of backwater curves, the water is flowing at the upper gradient (see Section 14), the velocity is low, and air entrainment can be neglected. The general case indicated in part (a) applies directly, except that for Eq. 51a the angle θ is usually so small that cos θ can be assumed equal to unity. The actual value of sine θ, of course, must be used. Also the velocity is small enough that a value of α of unity is close enough. That is,

$$d' = l \sin \theta + d + \frac{v^2 - v'^2}{2g} - lS \qquad [57]$$

The river is divided into a number of reaches as shown in Fig. 35, although the reaches are much more numerous than indicated there. Each reach should

* See Ref. 27 of Section 19, pp. 1407-1408 and p. 1482.

include a length that is reasonably uniform in section and shape. The more numerous the reaches the more accurate will be the determination of the slope.

Starting with the section at the dam, where the value of d is known, a value of d' for the other end of the reach is tentatively assumed. With these values of d and d' the values of the hydraulic radius and velocity at each end of the reach are calculated and averaged. With these average values and the proper friction coefficient n, the slope and the value of lS are obtained from Eq. 16*a*, using Fig. 12, adding the necessary losses for bends, obstructions, and sudden contractions and enlargements.

From Eq. 57 a value of d' is calculated that should agree with the one tentatively assumed. If it does not, then a new value of d' should be tentatively assumed and the calculations repeated until an agreement is reached.

Values so determined for the upper end of the first reach should be used for the lower end of the second reach.

When a dam is built on a flat, silt-laden stream, silt deposits not only in the reservoir but also in the river above the reservoir, raising its bed so high that high water is increased far beyond the backwater curve as computed for unsilted water. This condition is controlled by many variables, and rules for its general solution are not available.

17. Hydraulic Models and Similitude.* (*a*) *Introduction.* Hydraulic model studies are finding increasing application for checking and modifying the analytical designs of hydraulic structures. Hydraulic models are usually constructed so that the scale ratio (L_r) of prototype to model is from 10 to 100. The model test provides a preview of the behavior of the structure under operating conditions, and it gives results of both qualitative and quantitative value.

A number of well-equipped hydraulic laboratories staffed with men of experience in the field of hydraulic experimentation are now available, and, as there are many pitfalls for the novice in this field, the more important hydraulic model testing should be intrusted to such laboratories.

(*b*) *Criteria for Similarity.* The criteria for similarity are dimensionless ratios of physical quantities, the terms depending on the type of forces acting. Being dimensionless, the absolute values of these ratios will not be affected by the system of units adopted in analysis. If two systems (such as the model and prototype) are to be dynamically similar, the ratio must have the same absolute value in each.

These criteria are

1. Weber's number for surface tension, W.
2. Cauchy's number for elasticity, C.
3. Reynolds' number for viscosity, R.
4. Froude's number for gravity, F.

* Contributed by George E. Barnes, Professor of Hydraulic and Sanitary Engineering, and Head, Department of Civil Engineering and Engineering Mechanics, Case Institute of Technology.

Since the effect of surface tension and elasticity is negligible for model studies in connection with hydroelectric projects, Weber's and Cauchy's numbers will be disregarded in this text.

In most such model tests the influence of viscosity is negligible but each case should be checked for the Reynolds' number to see that the model flow will not be in the laminar range but in the turbulent range. Reynolds number is

$$R = \frac{V_m L_m p}{u} \quad [58]$$

where V_m = model velocity in feet per second

L_m = any model length parameter influencing flow (diameter, hydraulic radius, etc.) (Note that L_m is usually diameter. Where other length functions such as head, length, etc., are used, the value of R will differ.)

p = mass density of fluid (w/g) for water at 60 degrees Fahrenheit $62.4/32.2 = 1.94$ slugs per cu ft,

u = coefficient of viscosity.

For practical purposes p/u may be assumed to be equal to 82,200 sec-ft^2 and R to be equal to at least 2500. Thus the value of $V_m L_m$ should not be less than 0.0304 if the flow is to be turbulent.

In all such studies the Froude number must be used. It is

$$F = \frac{V_m^2}{gL_m} \quad [59]$$

where g = acceleration of gravity = 32.2 ft per second per second.

(c) *Significance of the Froude Number.* With due allowance for friction effects and for the ordinary range of studies in free fall, flow over spillways, open channel flow, flow in intakes, penstocks, etc., virtual dynamic similarity obtains when the Froude number is the same for model and prototype, because only gravitational and inertia forces are operative in any appreciable degree.

Assuming that in a given case the Froude number is the criterion for similarity, an immediate algebraic consequence is to establish implicit relationships between quantities in the prototype and homologues in the model, as follows. Let V_1, Q_1, A_1, R_1, S_1, L_1, D_1, n_1 and other common symbols denote velocity, discharge, area, hydraulic radius, slope, length, diameter, roughness coefficient, etc., for the prototype and let V_m, Q_m, A_m, R_m, S_m, L_m, D_m, n_m denote corresponding terms in the model. Let L_r denote the scale ratio of prototype to model. Then, if F (the Froude number) is the same for model and prototype, the following relations are readily derived for dynamic similarity with respect to gravitational forces: units of length (head, hydraulic radius, diameter, length, depth, width, etc.)

Froude number	$F = \dfrac{V_p^2}{gL_p}$
Length	$L_p = L_m(L_r)$
Time	$T_p = T_m(L_r)^{1/2}$
Velocity	$V_p = V_m(L_r)^{1/2}$
Areas	$A_p = A_m(L_r)^2$
Discharge	$Q_p = Q_m(L_r)^{5/2}$
Slopes, accelerations, constants	$C_p = C_m(L_r)^0$
Power	$P_p = P_m(L_r)^{7/2}$
Work	$E_p = E_m(L_r)^4$

(*d*) *Coefficient of Roughness in Model Tests.* Where roughness of channel plays a large part, as in rivers, it becomes necessary to determine a roughness coefficient for the model that will make it dynamically similar to the prototype. Roughness ratios, determined from the well-known hydraulic formulas given in this chapter, are commonly used.

Using Manning's formula, $V = 1.486R^{2/3}S^{1/2}/n$, and, making the slopes in prototype and model equal, we find that

$$n_m = n_p\left(\frac{1}{L_r}\right)^{1/6} \qquad [60]$$

where n_m is coefficient of roughness in model;
n_p is coefficient of roughness in prototype.

A similar relation can be obtained for any of the other formulas indicated in this chapter as being applicable.

(*e*) *Example.* An open rectangular flume with shooting flow is to form the hydraulic jump. (Width 10 ft, depth before jump 2.0 ft, velocity before jump 40 ft per second.) The hydraulic jump formula,

$$D_2 = \sqrt{\frac{2V_1^2D_1}{g} + \frac{D_1^2}{4}} - \frac{D_1}{2}$$

in which D_1 and V_1 are depth and velocity before jump and D_2 is depth after jump, is based upon gravitational forces only. If the Froude number for similarity is used, what will be the characteristics of the jump in the model on a scale of 16?

By the above formula, $D_2 = 13.1$ ft. $Q = 10 \times 2 \times 40 = 800$ sec-ft.

$$F = \frac{V_1^2}{gD_1} = \frac{40^2}{32.2 \times 2.0} = 24.8$$

which also holds for the model in

$$F = \frac{[40(1/L_r)^{1/2}]^2}{32.2 \times 2(1/L_r)} = 24.8$$

where L_r is the scale ratio.

By the relationship previously stated, the model will have the following dimensions:

$$D_1 = 2.0\left(\frac{1}{L_r}\right) = \frac{2}{16} = 0.125 \text{ ft}$$

$$V_1 = 40\left(\frac{1}{L_r}\right)^{1/2} = 40\left(\frac{1}{16}\right)^{1/2} = 10 \text{ ft per second}$$

$$Q = 800\left(\frac{1}{L_r}\right)^{5/2} = 800\left(\frac{1}{16}\right)^{5/2} = 0.781 \text{ sec-ft}$$

$$D_2 = 13.1\left(\frac{1}{L_r}\right) = 13.1\left(\frac{1}{16}\right) = 0.82 \text{ ft}$$

Incidentally, the above model values for D_1 and velocity, if substituted in the hydraulic jump formula, will give $D_2 = 0.82$ ft, thus verifying the similarity condition.

Flow in the prototype will obviously be turbulent. Flow in the model must likewise be turbulent if V_mL_m exceeds 0.0304. To check this, compute V_mL_m, using $4r$ (4 times the hydraulic radius in the model) as the equivalent of L_m. This is necessary, because $R = 2500$ for turbulent flow is based on flow in a circular section where L is the diameter of the circle and equals $4r$. Thus,

$$V_mL_m = 10 \times 0.358 = 3.58$$

Since 3.58 exceeds the required value of 0.0304, flow in the model will be turbulent, and Reynolds' number does not apply.

18. The Flow Net. Hydraulic problems involving varying velocities and consequently pressures deviating from the conventional hydrostatic law can be solved by means of a flow net. As indicated in Fig. 36, a flow net is a graphical representation of the flow conditions and consists of stream lines and equipotential lines. The stream lines, called ψ lines in Fig. 36, are conjugate functions of the equipotential lines, or ϕ lines. In form and construction this flow net is similar to that for seepage through porous mediums such as earth foundations, as explained in Section 4c, Chapter 25.

The procedure for constructing the flow net in Fig. 36, according to Lane, Campbell, and Price [36], is as follows.

Step 1. Extend the upstream face of the dam upward to intersect the headwater level at point O. With point O as a center and with a radius of 2 or 3 times the head on the dam, draw the arc of a circle representing $\phi = 0$ between the upstream face of the dam and the water surface.

Step 2. Divide the equipotential line, $\phi = 0$, into a number of equal parts. Beginning at this line, roughly sketch the entire flow net with curvilinear squares, guessing the boundaries of flow as accurately as possible.

Step 3. By a cut-and-try procedure adjust the flow net until it satisfies the criteria that the length and width of each square are equal and that all lines intersect at right angles.

Step 4. Establish a scalar value of velocity by computing the surface velocity at any point A, from $v = \sqrt{2gh_a}$. Solve for the pressure, at any point, by Bernoulli's Theorem.

For a detailed description of methods of constructing and solving the flow net, see Refs 36 and 37 of Section 19.

The employment of the flow net is limited to situations where friction and eddy losses do not have a controlling effect on the velocities or pressures. In two-dimensional flow, as in flow over straight spillway dams, the flow net has produced good results. In three-dimensional flow, however, as for a "morning-glory" jet, the flow net gives approximate results and should serve mainly as a guide for the construction and testing of a model

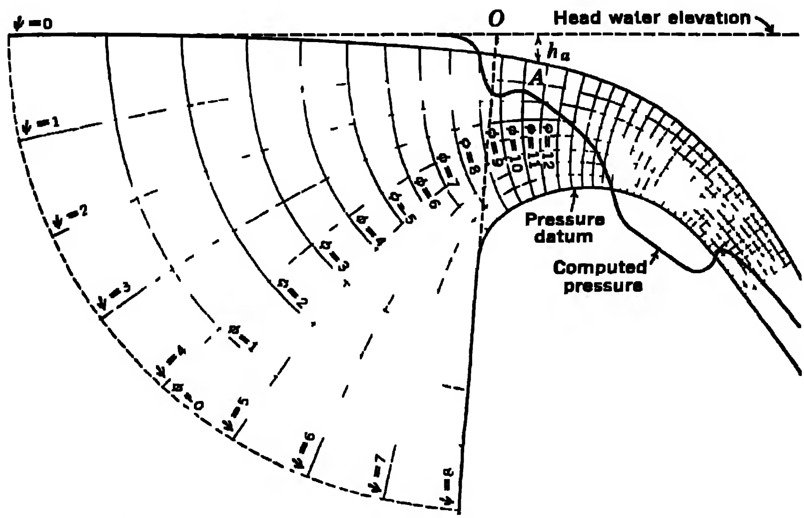


FIG. 36. Flow net for two-dimensional flow over a dam (*Civil Eng* October 1934, p 510)

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CHAPTER 9

HEAD, POWER, AND EFFICIENCY

1. Head. The *gross head* on a hydroelectric plant is the difference between headwater elevation and tailwater elevation when the plant is not in operation. The gross head varies with the conditions of flow in the stream and may be very different at time of minimum stream flow from what it is when the stream is in flood. Unless qualified, the usual or normal gross head is understood.

In considering the economic advisability of a project, it is important to know what variation in gross head may be expected through the years and also the extreme variation from minimum to maximum gross head. Thus a low-head plant on a river subject to great floods, such as the Ohio, may have such a small minimum gross head that operation of the plant will be impaired or the turbines be entirely unable to operate at time of great floods, in which event the plant would have little or no firm capacity (see Section 3, Chapter 15).

Only a part of the gross head can be utilized in producing power. There are losses of head in the penstock and conduits or headrace supplying the turbines, losses in water passages of the power plant, and losses in the tailrace.

The difference in the elevation of water surface in a tailrace for no plant discharge and for full plant discharge with all water flowing through the plant may be from 4 to 12 ft. With a low-head plant this loss may be a substantial percentage of the head available.

Within limits all these losses are controllable, since they decrease with increasing size of water passages. For any given set of conditions, there is a point where additional expenditures for increasing the size of these water passages will no longer produce a satisfactory return on the additional investment. Thus it is one function of the engineer to determine what head losses it is economical to incur for a proposed hydroelectric project.

The *net head* or *effective head* on a hydroelectric plant is the gross head minus all losses above entrance to the scroll case and below exit from the draft tube. The American Society of Mechanical Engineers' Test Code for Hydraulic Prime Movers (1938) defines effective head as "The total net height of the water column effective on the turbine runner when generating power" (see Fig. 1).

Net head varies with the gross head, and in addition it varies with the load on the unit because the friction and velocity losses increase approximately as the square of the discharge. The net head at full load for low-head devel-

opments with very short or no conduits or tailrace may be nearly as much as the gross head; but for high-head developments, requiring long conduits, the net head may be only 90%, or even less, of the gross head. Unless otherwise qualified, the term net head is understood to mean the net head, full plant discharge under normal or usual stream-flow conditions. Minimum net head limits the firm capacity of the plant (see Section 3, Chapter 15).

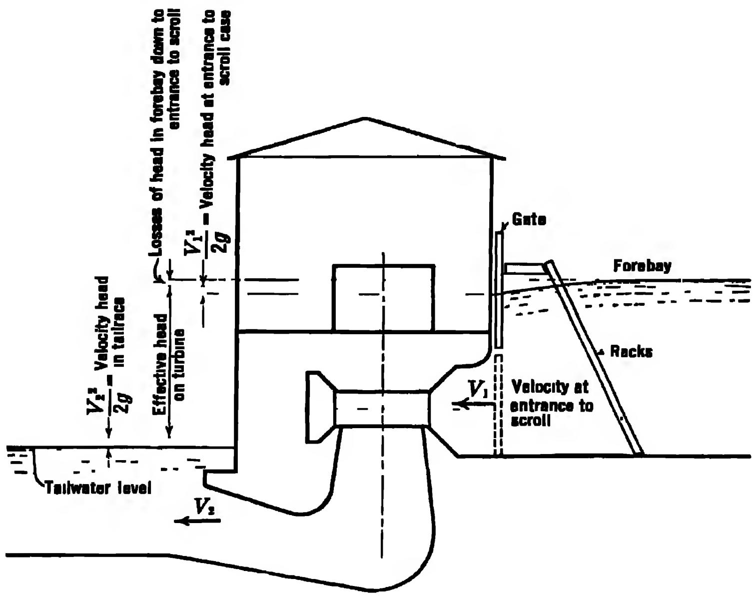

FIG. 1. Effective head on a hydro plant.

Drawdown for pondage varies with the topographical features at the site and with the nature of the load. For low-head developments, the average drawdown required is not often in excess of 5% of the total head, and for very high-head developments it is usually negligible.

If the lake at the dam is to be used for storage, the extent of drawdown may be a large percentage of the total head, depending upon the relative value of water and head. It is evident that the last of the water from the reservoir is of no practical value if all the available head is used up to provide it. Special cases of excessive drawdown are those in which the turbines are installed at storage reservoirs as supplementary units to other developments for which the reservoir is regulated.

The productive head is that which, when combined with average discharge, gives average output. The net head corresponding to average dis-

charge is greater than the productive head and, if used, will give too large a calculated output, as explained in Section 2.

2. Energy, Work, and Power. *Energy* is the ability to do work. Work is the utilization of energy and is equal to force times distance. Both energy and work are measured by the same unit; the foot-pound (1 lb raised 1 ft) being the primary unit in the English system. In hydroelectric work the kilowatt-hour is the usual unit of energy and equals approximately 1,476,000 ft-lb. The potential energy of a volume of water is the product of its weight by the head, or vertical distance it may be lowered. In the transformation of this energy into useful work, part will be lost in the conduit system and part in the apparatus of the development.

If a volume of water in a storage reservoir is allowed to pass through the turbines under a constant net head, the energy of this water delivered in the form of work is

$$K = wV(H - h_f)e \tag{1}$$

where K = energy in foot-pounds;
w = the weight of 1 cu ft of water in pounds;
V = the volume of water, in cubic feet;
H = the gross head, in feet;
h_f = the head lost in the conduit system and tailrace;
e = the station efficiency, expressed as a fraction.
Also let q = the discharge of a stream in cubic feet per second;
t = a period of time in seconds;
T = a period of time in hours;
p_H = horsepower;
p_K = kilowatts.

If the discharge is passed through the turbines, the total volume V used during the period t is

$$V = qt$$

which, if used in Eq. 1, gives

$$K = wqt(H - h_f)e \tag{2}$$

which is the delivered energy of the stream during the period t.

Power is the rate of work, or the work done in a specified time. The usual units of power are the horsepower (hp) and the kilowatt (kw) these being, respectively, 550 and 737 ft-lb per second. The power p corresponding to K ft-lb of energy in time t is

$$p_H = \frac{K}{550t} \quad \text{horsepower} \tag{3}$$

or

$$p_K = \frac{K}{737t} \quad \text{kilowatts} \tag{4}$$

Substituting the value of K from Eq. 2, and using the usual value * of 62.5 lb in place of w, the expression for power is

$$p_H = \frac{q(H - h_f)e}{8.8} \tag{5}$$

or

$$p_K = \frac{q(H - h_f)e}{11.8} \tag{6}$$

Energy may be expressed in horsepower-hours (hp-hr) or kilowatt-hours (kw-hr), and Eqs. 5 and 6 may be converted into

$$\text{hp-hr} = p_H T = \frac{q(H - h_f)eT}{8.8} \tag{7}$$

$$\text{kw-hr} = p_K T = \frac{q(H - h_f)eT}{11.8} \tag{8}$$

where T is the period of flow in hours.

3. Determination of True Output and Average Power. Equations 5 to 8, inclusive, are the basic expressions used in calculations for power and

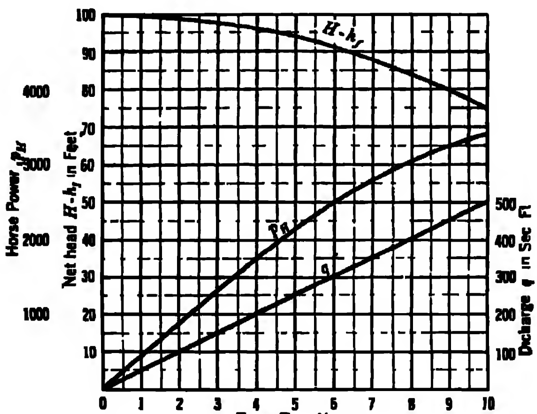

FIG. 2. Net head and power for case in which discharge varies from 0 to 500 sec-ft at end of 10-hr period.

energy. Equations 5 and 6 are correct for power at any instant, and Eqs. 7 and 8 are correct only if q is constant during the period T. If q varies during the period, h_f also varies, but not in direct proportion because it is approximately proportional to the square of the conduit velocities. Therefore, as indicated below, the net head, $H - h_f$, corresponding to average discharge, cannot be used in Eqs. 5 and 6 to calculate average power or in Eqs.

* The weight of 1 cu ft of water varies, for all practical purposes, between very narrow limits, and for ordinary computations may be taken as 62.5 lb per cu ft.

7 and 8 to calculate energy output during a given period, unless the friction head, h_f, is very small or q is practically constant.

In Fig. 2, values of q, $H - h_f$, and p_H are indicated for a case where q varies from 0 to 500 sec-ft at the end of a 10-hr period. The equations are

$$H - h_f = 100 - \frac{q^2}{10{,}000}$$

and

$$p_H = \frac{q(H - h_f)e}{8.8} = \frac{q(H - h_f)}{11}$$

efficiency, e, being assumed equal to 80%.

The average power during the period, calculated from the curve, is 1990 hp. The average discharge during the period is 250 sec-ft. The net head, corresponding to average discharge, is seen to be 93.75 ft. If this value of net head were used to determine the average horsepower during the period, the erroneous result would be

$$p_H = \frac{250 \times 93.75}{11} = 2130 \text{ hp}$$

which is seen to be considerably greater than the true value of 1990.

Calculations necessary to determine the true average output, as indicated for Fig 2, are too laborious for practical use, and an approximate correction is made by multiplying the head corresponding to average discharge by a coefficient, C, or, letting

P_H = average horsepower during a given period;
P_k = average kilowatts during the period;
Q = average discharge during the period;
H_f = conduit losses corresponding to Q.

Then

$$P_H = \frac{Q(H - CH_f)e}{8.8} = \frac{Qhe}{8.8} \qquad [9]$$

$$P_k = \frac{Q(H - CH_f)e}{11.8} = \frac{Qhe}{11.8} \qquad [10]$$

$$\text{hp-hr} = \frac{Q(H - CH_f)eT}{8.8} = \frac{QheT}{8.8} \qquad [11]$$

$$\text{kw-hr} = \frac{Q(H - CH_f)eT}{11.8} = \frac{QheT}{11.8} \qquad [12]$$

$(H - CH_f) = h$ is termed the *productive head*. It is always equal to or smaller than the net head corresponding to Q. In the preceding example, $C = 2$ and the productive head is $100 - 2 \times 6.25 = 87.5$ ft as compared with the head, 93.75 ft, corresponding to average Q.

The value C is theoretically obtained by dividing the total period into a number of equal intervals of time so small that the discharge is practically constant for each interval.

Let q_1, q_2, etc. = the average discharge during each interval;
n = the number of intervals.

Then

$$C = \frac{q_1^3 + q_2^3 + \cdots q_n^3}{nQ^3} \qquad [13]$$

In practice, however, the load curve or variation in demanded power output is first known and Q is desired. In such cases, C may be approximately determined by substituting power in place of discharge in Eq. 13.

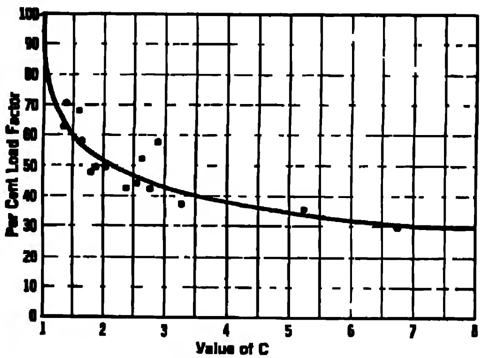

FIG. 3 Values of C in Eq 13

C tends to increase as the load factor decreases. An approximate relation between C and load factor is indicated in Fig. 3, in which the plotted points were computed from a number of typical load curves of public-utility plants. The error involved in the use of Eqs. 5 to 8 inclusive is not on the side of conservatism but will ordinarily be negligible if the conduit losses are small and will not exceed about 4% unless the head lost in the conduit system at full load is in excess of about 10% and the load factor less than 30%. Therefore, the use of Fig. 3 to determine the coefficient C for use in the more accurate Eqs. 9 to 12 inclusive will ordinarily give results well within the degree of precision desired.

4. Efficiency of Apparatus. The unavoidable losses in the generation and transformation of the energy of a water-power development, and in its transmission to the point of ultimate use, frequently amount to a large part of the total energy available. As already indicated, the losses in the conduit system are accounted for in the use of net or productive head at the turbines and are not charged against the efficiency of the station. The energy lost in the various types of apparatus required for the generation of mechanical energy by the turbine, its conversion to electrical energy by the generator,

its transformation to high voltage suitable for transmission, its transmission, and its transformation to low voltage suitable for local distribution is measured by the efficiency of the apparatus.

Efficiency is the ratio between the power delivered by a machine or other apparatus and the power supplied to it, usually expressed as a percentage. Thus, if 8000 kw is being supplied to a turbine by the water passing through it and 7000 kw is delivered to the generator shaft, the efficiency of the turbine is 87.5%. If the generator then delivers 6500 kw to the bus-bars, the efficiency of the generator is 93%. The combined efficiency of several machines is the product of their individual efficiencies. In the preceding case, the combined efficiency of the turbine and generator is 81.5%. The efficiency of the station is generally considered as the ratio between the energy delivered to the low-tension side of the step-up transformers and the energy supplied to the turbine, and is the product of the efficiencies up to that point.

This facilitates comparison with steam plants because, for steam plants, the output is generally recorded as at the low-tension side of the step-up transformers. Frequently, however, the steam plant is located near the center of the market, whereas the hydro plant may be remote from it. In order to put the hydro output on a comparable basis, then, the capacity and output of the hydro plant must be reduced by the losses of transformation and transmission to some location which is as favorable for distribution as that of the comparable steam plant.

The efficiency of the various forms of apparatus used in a hydroelectric system is not constant but changes with the load. The variation of efficiency with power depends upon the characteristics of the apparatus and is often determinable only by tests. Typical efficiencies of apparatus and transmission-line efficiencies for a single-unit hydroelectric station are given in Fig. 4 and for a three-unit plant in Fig. 5. It will be noted that, with the exception of the transmission line, the efficiency for small percentages of full load is low for the single-unit plant.

However, the above is not usually a sound reason for refusing to consider a single-unit plant. Modern power systems generally include a multiplicity of generating units, both steam and hydro. Consequently, a single-unit plant, possessed of pondage, may generally be operated at or near the point of maximum efficiency or not at all. There is frequently a material over-all saving in installing a single medium-sized unit instead of several very small units of the same total capacity.

Actual efficiencies of apparatus are frequently found to be several per cent higher than guaranteed, but it is not safe to count on this possibility. The efficiency of turbines, generators, and other types of apparatus varies somewhat with their size. Small units may have slightly lower, and large units higher, efficiencies than those shown in Figs. 4 and 5. The maximum efficiencies so far obtained for very large turbines and generators are about 94 and 97%, respectively, at points of best load.

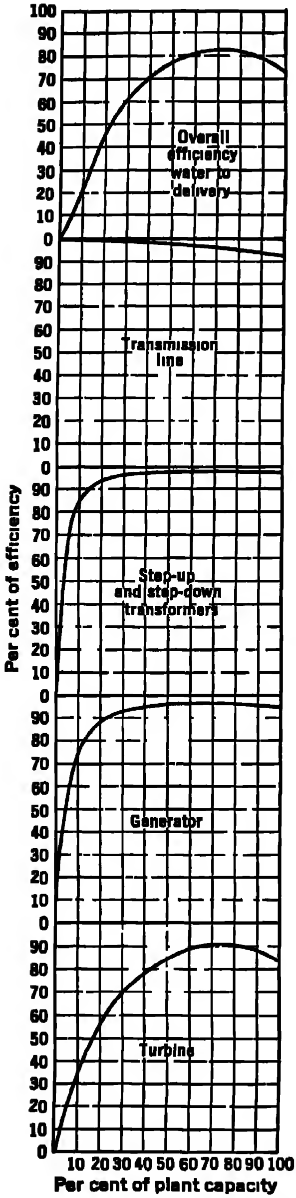

FIG. 4 Efficiency of typical single-unit hydroelectric development.

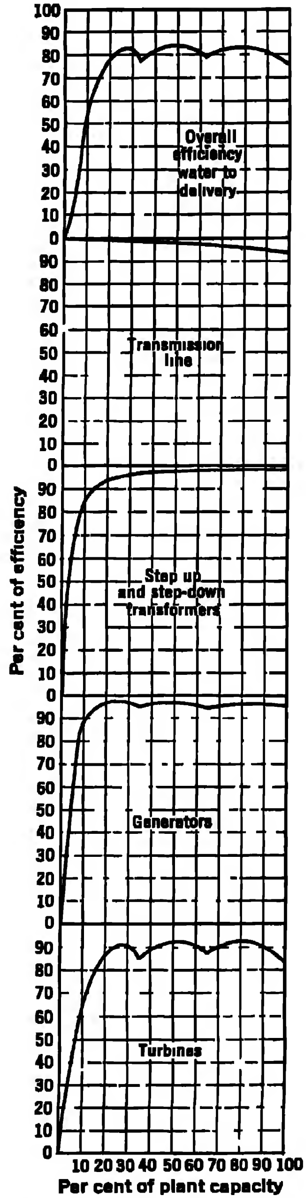

FIG 5 Efficiency of typical three-unit hydroelectric development.

Transmission-line efficiency is seldom less than 90% even for long lines; but, other things being equal, long lines will have lower efficiency than short lines (see Chapter 43). The efficiency of the 60-mile, 220-kv Conowingo line, for instance, is approximately 98%, and over-all efficiency from water to input side of transformers at terminal is approximately 79%.

For detailed information on turbine efficiency see Section 8, Chapter 38, and for generator and transformer efficiency see Sections 5 and 37 of Chapter 41. For the variation of turbine efficiency with changes in head, see Section 17 and Fig. 24 of Chapter 38.

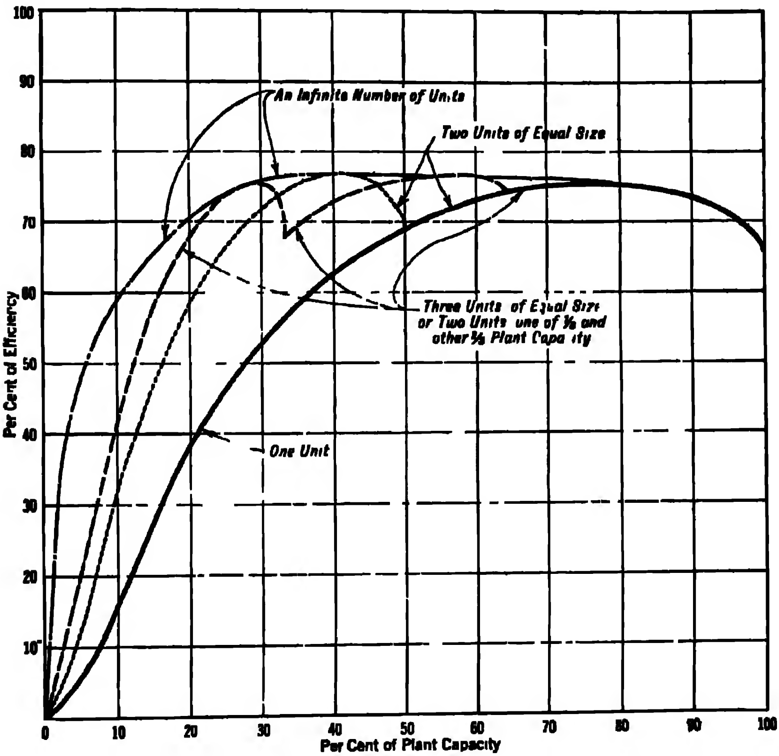

FIG. 6 Effect of number of units on over-all plant efficiency.

5. Effect of Number of Units on Plant Efficiency. Figure 6 shows a typical comparison of over-all efficiencies between plants containing the following combinations of similar units:

1. A single unit.
2. Two units of equal size.
3. Three units of equal size.
4. Two units, one of one third and the other two thirds of station capacity.
5. An infinite number of units.

The scheme providing two units of unequal size (4) is an unusual type advocated by very few engineers. It has the same efficiency as a three-unit plant, but it is probably not much, if at all, less costly than a three-unit plant and has the objection of noninterchangeability of parts.

It is seen in Fig. 6 that a two-unit plant has decidedly better average efficiency than a single-unit plant, but the advantage of a three-unit plant over a two-unit plant is less marked. It is interesting to note the rather small difference in efficiency, for 30 to 100% of station capacity, between a three-unit plant and a theoretical plant containing an infinite number of units. In a hydro plant which has pondage and which is part of a large interconnected system, the number of units in the plant makes very little difference in over-all efficiency. In effect, the entire system is a single plant and nearly all the units are operated at point of maximum efficiency or not at all. Without pondage or with deficient pondage, the efficient utilization of the available water supply may make advisable the use of a number of units even though the size of these units may be smaller than would otherwise be economical.

6. Average Over-all Plant Efficiency. The output and capacity of steam plants is generally stated as at the low-tension side of the step-up transformers. Correspondingly, the output capacity of hydro is taken at the same point and over-all plant efficiency (or station efficiency) is measured from water to the low-tension side of the step-up transformers (see Section 1 of this chapter).

Many large modern hydroelectric plants have an average over-all plant efficiency of 80 to 83%. This and efficiencies given in Table 1 are based on

TABLE 1

TYPICAL AVERAGE EFFICIENCIES OF MODERN HYDROELECTRIC PLANTS WHICH ARE A PART OF LARGE POWER SYSTEMS

	Per Cent Efficiency	Per Cent Accumulative Efficiency
Turbines	87.0	87.0
Generators (including excitation and station use)	95.0	82.6
Step-up transformers	98.0	81.0
Transmission line	96.0	77.9
Step-down transformers	98.0	76.3
Over-all plant efficiency	....	82.6

net head in accordance with usual practice. If based on gross head the efficiencies stated would, in general, be from 10 to 15% less. Station service use of power which is included in deriving the above is generally less than 1%.

As the load, and hence the efficiency, is not constant, the average station efficiency for use in computing energy available from stream flow depends upon the shape of the load curve, the apparatus employed, and the variation

in head For units of more than 5000-kw capacity, having a variation in head not in excess of about 10%, Table 1 gives conservative values of average efficiency for a plant having two or more units generating for a typical nearby general city load on a large system

The efficiency of turbines decreases with age only if proper maintenance and repairs are neglected The efficiencies of generators and transformers are practically permanent if the insulation is not allowed to deteriorate Transmission-line efficiency changes only by reason of damage to the wire or insulators or by short-circuiting Under ordinary conditions no allowance need be made for reductions in efficiency with age, if there is ample opportunity for thorough inspection at frequent intervals and for proper maintenance

For efficiency tests on hydroelectric plants, see Section 22, Chapter 38 The variation of turbine efficiency with changes in head is given in Section 17 and Fig 24, Chapter 38

CHAPTER 10

PONDAGE AND STORAGE

1. Pondage. A hydro plant is said to have ample pondage if the capacity of the pond above the intake is sufficient to take care of the hour-to-hour fluctuations of the load on the plant throughout the period of 1 week (see also Section 4, Chapter 11). Some ponds, though large enough to take care of some load fluctuation, cannot do so throughout a period of a week during time of minimum stream flow. The plant is then said to have deficient pondage.

Low-head run-of-river plants on navigable streams, such as the Ohio, the Tennessee, and the Kanawha, usually have deficient pondage or none at all because pond level must be maintained within narrow limits at a predetermined level. With no pondage, the firm capacity (Section 3, Chapter 15) of the development could be equal only to the power which the minimum flow of the stream would produce, whereas with ample pondage the installation might advisedly be based on 10 to 15 times the minimum flow (see Chapter 15).

2. Effect of Pondage on Plant Capacity. The advisable capacity of a hydroelectric project depends upon the extent to which the installation may be utilized within the limits of the connected load curve (see Chapter 14) The following examples will serve to indicate the various limitations of a water-power development as affected by market requirements, capacity, pondage, stream flow, and steam plants.

Case 1. Plant capacity equal to peak demand
- (*a*) With adequate pondage
- (*b*) Without adequate pondage

Case 2. Plant capacity less than peak demand.
- (*a*) With adequate pondage.
- (*b*) Without adequate pondage.

Case 1*a*. In Sketch *A* of Fig. 1 the uniform stream flow, reduced to kilowatts, is shown slightly greater than that corresponding to average power demand.

Assuming the pond to be full at 8:15 A.M., a portion, *P*, of the load between 8:15 A.M. and 10:40 P.M. is generated from water drawn from pondage. Between 10:40 P.M. and 3:30 A.M., the excess flow, *F* (equal to *P*), not required for power, is used to refill the pond, and the remainder, *L*, of the excess flow must be wasted over the dam.

In Sketch *B*, the flow is shown exactly equal to that required for average demand. The pond is drawn down between 7:40 A.M. and 11:00 P.M. and refilled during the remainder of the day. As the flow corresponds to average

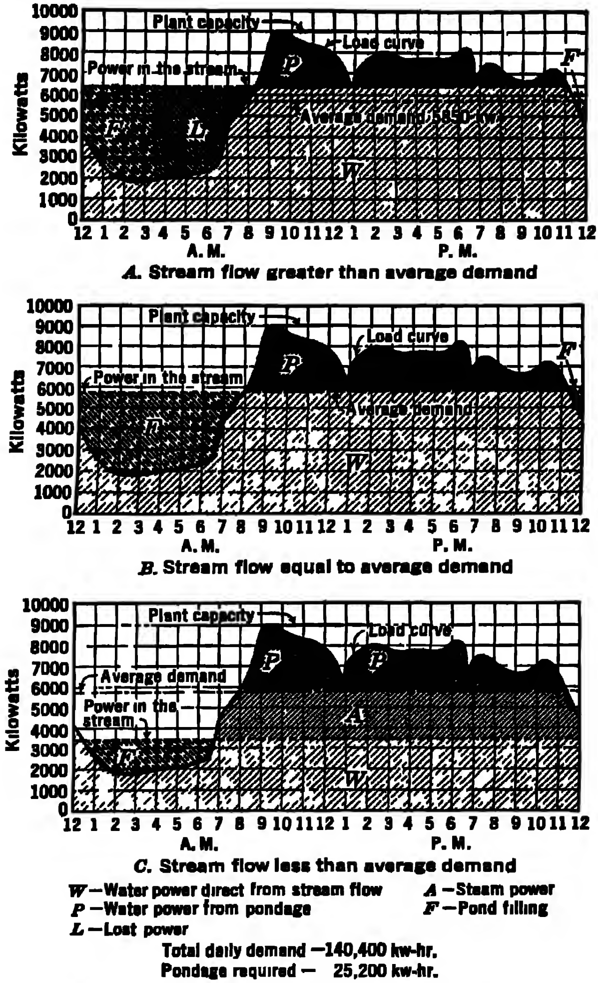

FIG. 1. Typical distribution of load. Plant capacity equal to peak demand and adequate pondage.

demand, no water is wasted, and the required draft from the pond is greater than for any other flow, for this case corresponding to an output of 25,200 kw-hr.

In Sketch *C*, the flow is less than that required for average demand; consequently a deficiency of water-power output exists and steam is called upon

to supplement the water power. The early morning excess flow, F, is retained in the pond and is used for the peak load, P. Steam supplies the block of power A.

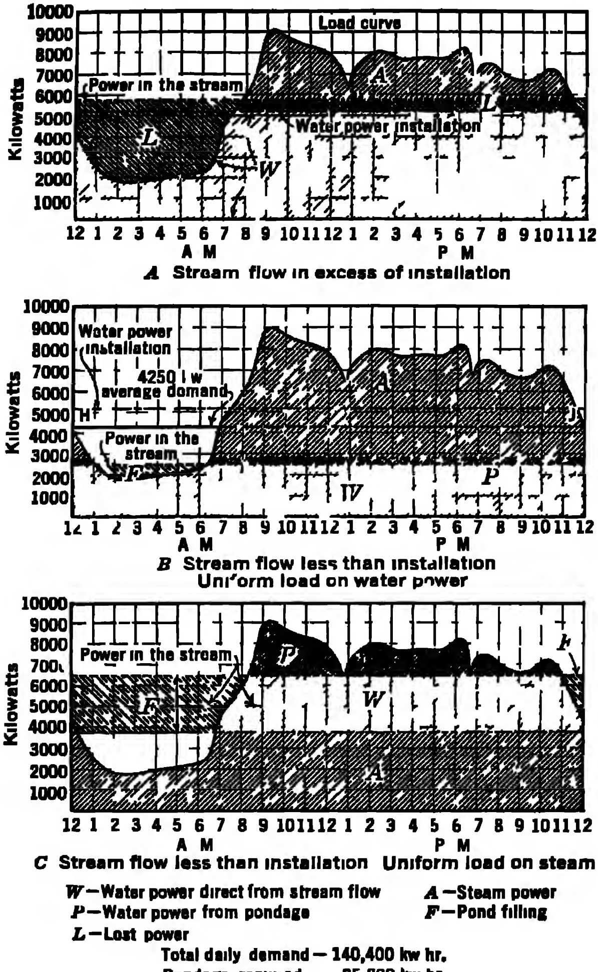

FIG. 2. Typical distribution of load. Plant capacity less than peak demand and adequate pondage.

It will be noted that, with adequate pondage, no deficiency of water-power output occurs except when the stream flow is less than that corresponding to average demand and that the average deficiency is equal to the difference between the power in the stream and the average demand.

Case 1b. If no pondage is available, the flow cannot be regulated, and all demanded load, above the "power in the stream" line of Sketches *A*, *B*, and *C*, of Fig. 1, must be supplied by steam or by some other source.

Case 2a. As indicated in Fig. 2, the installation is inadequate to the peak demand. In Sketch *A* the power in the stream is shown to be in excess of installation. No pondage is utilized, and all flows, *L*, above the load curve and the installation capacity are wasted. Steam would be called upon for the load *A*.

If the power in the stream is less than plant capacity, as indicated in Sketch *B*, the excess early morning flow, *F*, is ponded and utilized between 6:15 A.M. and 12:45 A.M., the whole flow being utilized for power. For this distribution of the load, the steam plants would be required to provide the deficiency, *A*, of output. A better distribution of loading, however, is possible. Steam plants operate with best efficiency on a uniform load. Therefore, with adequate pondage, the distribution of load shown in Sketch *B* is less advantageous than that indicated in Sketch *C*, where the flow is utilized to allow the steam to operate on a more uniform output. The stream flow, *F* during the early morning, is held in the pond and utilized during the day to carry the peak of the load, *P*. This distribution requires considerably more pondage than that shown in Sketch *B*. In both cases, the whole flow is utilized.

From Fig. 2 it can be calculated that the average demand below the water-power installation is 4250 kw. Therefore, the line *HJ* of Fig. 2 indicates, for this example, the maximum possible output from water power with ample stream flow.

Case 2b. Without pondage, the water-power output capacity, for flows l[illegible] than 4250 kw, is exactly the same as for Case 1*b*.

A daily load curve has been used in the foregoing discussion; but a similar study, based on a weekly load curve as indicated in Fig. 8, Chapter 14, must be used if the Saturday and Sunday loads are different from those of weekdays. Moreover, studies are usually made for the four seasons of the year if the seasonal load is variable.

Also, for the sake of simplicity it has been assumed that the power supply system consists of the one hydro plant and a steam plant. For the discussion of factors leading to the determination of the capacity of a proposed hydroelectric project, see Chapters 14 and 15.

3. Flow Demand. The flow demand is the stream flow required to meet power demands. Thus, for a run-of-river plant which is part of a large power system, the flow demand is equal to the full plant discharge in second-feet. However, flow demand is not always equal to plant discharge. Thus, consider a plant with ample pondage, whose full plant discharge is 10,000 sec-ft, which is connected to a large load such that all the installed capacity will be firm capacity (see Section 3, Chapter 15). If it operates on a minimum capacity factor (see Section 8, Chapter 12) of 30%, the flow demand

will be 3000 sec-ft for the 24 hours, but the maximum flow demand will be 10,000 sec-ft (plant capacity).

4. Studies Involving Pondage, Storage, and Output. In computations for power and energy available from a hydroelectric development during a given period, it is customary to convert the number of kilowatts of power needed to meet requirements to its equivalent in second-feet of flow demand. The flow demand is then used as a basis for estimating storage requirements.

The application of hydrographs, mass curves, duration curves, and analytical methods to calculations of storage requirements and estimates of output will be given in this chapter. It will be seen that each of these methods has particular usefulness in special cases; but frequently the choice of method, though depending to a considerable extent on the nature of the problem, is dictated by the preference of the engineer and the manner in which the results of his calculations are to be published.

Methods suitable to four general cases are outlined briefly below. Where several methods are indicated, the first is usually the authors' preference. Pondage in excess of an amount sufficient to control the flow and make it conform to the varying daily and weekly demand is classed as storage.

Case 1. No storage, adequate pondage.
  Analytical methods.
  The hydrograph.
  The duration curve.
Case 2. No storage, no pondage.
  The duration curve.
Case 3. Storage at the plant, adequate pondage.
  Analytical methods.
  The mass curve.
  The hydrograph.
Case 4. Remote storage, adequate pondage.
  Analytical methods.
Case 5. Remote storage, no pondage.
  A combination of the analytical method and the duration curve.

5. Storage. In most highly developed streams, the natural flow during a large part of the year is deficient for the flow demand, and regulation by storage is desirable. There are relatively few streams whose flow is so well regulated by nature, owing to lakes or favorable soil conditions, that artificial storage is not desirable. Examples of such streams are the Niagara, the St. Lawrence, and the Au Sable and Manistee in Michigan. However, for most streams utilized for power, storage is desirable if obtainable at economic cost.

In general the principal economic function of storage is to increase the minimum flow of the stream and thus permit the installation of more capacity, which will be firm, in existing or projected plants down the river, i.e., capacity which will be just as useful on the connected load curve as alternative steam capacity (see Section 3, Chapter 15). The addition of storage

usually means that any given project will turn out more energy, but this consideration, although important, is generally of less total annual value than the increase in firm capacity. Complete regulation is seldom, if ever, economical, but partial regulation is often advisable, particularly where the water released from the storage reservoir will pass through a number of power plants aggregating a substantial head.

6. Silt Deposits in Reservoirs. The capacities of storage reservoirs are often affected greatly by the gradual deposition of silt. In some districts these deposits may be enormous and may, in the course of a few years, completely fill the reservoir and destroy its usefulness. Sluices in the dam are never effective in removing sediment, except that which has been deposited near the dam. Silting from forest-covered areas is negligible. In regions subject to violent rainstorms and not protected by vegetation, as in the Southwest, streams are heavily laden with silt.

Many tests of the silt content of streams have been made by the U. S. Reclamation Service and others. Records of 16 years of observations of the Rio Grande show an average of 1⅔% of silt carried in suspension, and several monthly averages in excess of 10%. Unfortunately, no satisfactory method has yet been devised for measuring the amount of sand, gravel, and even large boulders which are projected along the bottom. It is claimed that usually the percentage of matter carried in this manner is relatively small The nature of deposits against existing dams and natural obstructions serves to indicate the nature of the transported materials.

To measure the percentage of silt carried in suspension, a given volume of water is taken and evaporated, and the residue weighed. The dry weight of 1 cu ft of deposited silt varies with the nature of the material and has been found to range between 50 and 90 lb. However, samples can be taken from natural deposits and the weight accurately determined.

A check can be made by measuring the depth of silt in existing reservoirs on similar streams if the original bottom elevation is known. Proper allowance should always be made for estimated reduction in capacity from silting, either by providing excess capacity to retain the silt for a long period or by control of other storage sites for future use.

Field and laboratory tests indicate that, during times of flood, "high density" currents laden with silt travel nearly the length of the reservoir near the bottom. Although the studies are not yet conclusive, it seems that, with sufficiently large flood conduits near the bottom of the dam, much of this suspended silt could be passed through the dam at times of flood [7].

7. Benefits from Storage. Assuming the necessary demand for hydroelectric power, the availability of storage may transform an otherwise unpromising project into a highly desirable one, as the minimum stream flow may thereby be multiplied many times, and consequently the capacity that can be installed and be firm capacity on the connected load curve will also be increased.

If the storage benefits only a single plant, then the entire cost of such storage must be borne by that plant. For some plants with very high heads, the installation of storage is economically advisable even though the stored water goes through only one plant, but such cases are relatively rare.

Usually it is necessary for the stored water to pass through plants aggregating a substantial gross head in order to make the provision of storage pay. The fact that the benefits from storage may generally be fairly allocated on a

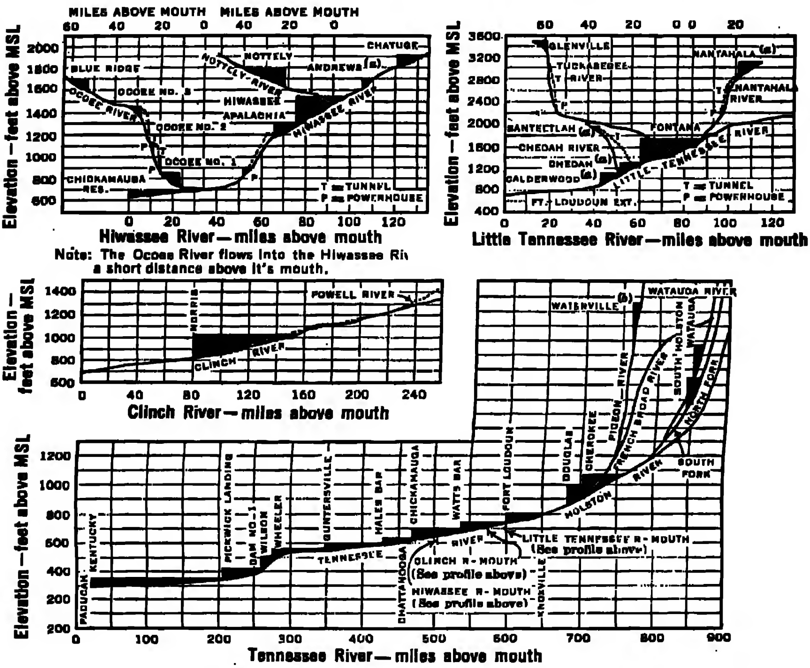

FIG. 3. Developments on Tennessee River and tributaries (see Table 1).

per foot of gross head basis has led the owners of power plants on a stream to get together and form associations or quasi-public regulating districts. The owners of the power plants contribute the necessary capital in proportion to their gross head, and storage reservoirs are constructed on the headwaters of the watershed. The owners then obtain the benefit from the stored water which is released during times of normally low stream flow and passed through their plants. The Black River Regulating District and the Hudson River Regulating District in New York State are illustrations of successful associations formed for the provision and utilization of storage.

On streams where all the hydro projects or riparian rights are owned by one or only a few interests, the development of storage is greatly facilitated. Figure 10, Chapter 11, indicates the manner in which storage and power have been coordinated by the Georgia Power Company on the Tallulah and Tugalo

TABLE 1

POWER AND STORAGE ON TENNESSEE RIVER AND TRIBUTARIES

A. Developments of Tennessee Valley Authority

(1) Project	(2) Watershed Area, sq mi	(3) Present Installation,* kw	(4) Gross Average Head, ft	(5) Power Storage, acre-ft
Kentucky	40,200	160,000	31	721,000
Pickwick Landing	32,820	144,000	50	239,200
Wilson	30,750	436,000	94	52,500
Wheeler	29,590	259,200	48	328,100
Guntersville	24,450	72,900	36	131,700
Hales Bar	21,790	51,100	38	13,100
Chickamauga	20,790	81,000	45	221,000
Watts Bar	17,310	150,000	56	214,100
Fort Loudon	9,550	128,000	70	79,300
Apalachia	1,018	75,000	433	35,700
Hiwassee	968	37,600	230	361,500
Nottely	214	. . . .	.	184,000
Chatuge	189	. .	. .	229,300
Ocoee No. 1	595	18,000	110	33,100
Ocoee No. 2	516	19,900	252	0
Ocoee No. 3	496	27,000	310	9,370
Blue Ridge	232	20,000	124	183,000
Norris	2,912	100,800	180	1,761,000
Fontana	1,571	135,000	425	1,157,000
Douglas	4,541	90,000	120	1,420,000
Cherokee	3,429	60,000	140	1,412,000
Total TVA plants		2,085,500		8,785,970

Note: Data for part A of Table 1 furnished by Tennessee Valley Authority.

B Developments of Private Companies on the Tributaries of the Tennessee River

Waterville	455	108,000	861	20,800
Cheoah	1,608	106,000	189	7,600
Santeelah	175	46,000	660	134,000
Calderwood	1,856	121,500	217	4,100
Nantahala	90	43,000	1,003	125,000
Glenville	37	22,000	1,215	66,800
Total private plants		446,500		358,300

* Includes installations under construction in 1948.

Note: The Waterville project is owned by Carolina Power and Light Company and the others by Aluminum Company of America or associated companies.

Rivers. Other examples of such coordinated systems of hydro power and storage are the Big Creek San Joaquin System of Southern California Edison Company, the Mokelumne River System of Pacific Gas and Electric Company in California, and the Catawba River System of the Duke Power Company in the Carolinas.

The Tennessee River and its tributaries furnish an outstanding example of the utilization of a large amount of storage in headwater reservoirs for increasing the firm capacity of down-river plants. The mean annual runoff of the watershed, in general, exceeds 1.6 sec-ft per square mile, but some of the headwater streams have a mean annual runoff exceeding 2.5 and even 3.0 sec-ft per square mile.

Before regulation, the flow of the main river was subject to very wide fluctuations, and the minimum flow had been as low as 0.08 sec-ft per square

TABLE 2

DAILY DISCHARGE, IN SECOND-FEET, OF TYPICAL RIVER FOR THE YEAR ENDING DEC. 31, 1941 (MINIMUM YEAR OF RECORD)

Drainage Area, 2000 Square Miles

Day	Jan	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1	3,200	3,850	10,600	6,000	8,700	1,100	650	810	4,500	3,560	1,710	3 015
2	3,350	5,500	10,200	6,400	8,500	1,200	670	1,040	5,100	4,500	1,830	3,500
3	3,500	12,400	10,300	6,000	7,100	600	1,050	1,660	5,500	5,100	2,070	3,700
4	3,700	10,200	9,500	5,300	6,800	900	1,000	1,970	5,700	4,900	2,380	3,900
5	3,800	8,500	10 700	5,400	7 000	1 400	1,210	1,610	5,600	6 000	2,490	3,800
6	3,650	5,300	11,700	5,700	6,900	1,600	1,200	1,600	5,700	7,000	2,370	3,550
7	3,500	4,900	12,000	5,200	6,400	700	1,340	1,980	5,650	6,100	2,180	3,770
8	3,400	4,900	11,900	5,300	6,200	800	1,590	2,250	6,000	5,000	2,340	3,880
9	3,450	4,800	12,000	5,000	5,800	750	960	2,210	6,400	6,000	2,410	4,000
10	3,650	4,850	12,600	5,200	5,900	850	1,010	1,920	5,000	4,400	2,500	3,850
11	3,640	4 500	12,100	6,000	3,400	700	1,130	2,000	5,100	4,500	2,170	4,050
12	3,510	4,550	12,500	7,200	4,000	680	1,200	1,860	4,800	4,100	2,380	4,480
13	3,500	4,300	12,350	8,300	4,500	650	1,210	1,170	5,300	3,120	2,200	4,500
14	3,540	4,220	12,600	8,200	3,800	620	1,100	1,520	5,600	3,180	2,110	4,620
15	3,710	4,230	12,800	8,250	3,000	600	1,080	1,570	4,260	2,500	1,900	4,850
16	4,000	4,100	12,300	8,250	2,500	550	900	2,040	3,000	2,180	2,020	4,300
17	4,240	4,700	12,000	10,000	2,600	650	900	2,400	2,770	2,100	2,170	4,150
18	4,100	4,900	12,200	9,800	2,400	710	780	2,710	2,850	2,220	2,250	4,200
19	3,920	5,200	10,800	10,300	2,600	920	780	2,500	2,970	2,470	2,160	4,000
20	3,950	5,000	12,300	10,200	2,700	670	1,010	2,660	2,800	2,610	2,200	3,580
21	4,100	7,000	11,700	9,700	2,200	620	960	2,840	2,870	2,150	2,300	3,570
22	4,250	8,500	11,750	9,300	2,300	640	970	3,010	2,880	2,270	2,370	3,520
23	4,320	9,600	11,500	8,500	2,400	700	920	3,270	2,900	2,090	2,330	3,180
24	4,250	8,700	10,500	8,400	2,000	850	1,000	3,380	2,890	2,010	2,390	3,500
25	4,200	9,100	8,100	8,500	1,600	1,030	1,150	3,540	2,500	1,400	2,390	3,610
26	4,150	10,000	5,900	9,500	1,700	1,310	1,370	3,750	2,200	1,570	2,380	3,870
27	4,080	10,500	6,000	8,800	1,000	1,600	1,280	3,800	2,490	1,480	2,740	4,000
28	3,950	11,000	6,200	9,000	1,500	1,000	1,140	3,900	2,370	1,600	2,710	3,850
29	3,960	...	6,400	8,900	1,200	780	980	4,000	2,000	1,380	2,780	3,760
30	3,850	...	6,550	8,800	1,000	680	800	4,100	2,850	1,570	2,940	3,700
31	3,800	...	6,700	...	1,100	...	800	4,200	...	1,550	...	3,570

mile at Florence, Ala., below Wilson Dam. The dams of the Tennessee Valley Authority on the main river, and particularly their headwater storage projects, have resulted in the partial regulation of the river for power and navigation. The headwater storage alone available for power exceeds 6½ million acre-ft; in addition, usable storage is available in the power projects and there is some incidental benefit to power from storage for flood-control purposes. The installation on the Tennessee River and its tributaries, including that of the Tennessee Valley Authority and that of privately owned plants, in 1948, exceeded 2 million kilowatts, with both the installation and the headwater storage expected to increase in the future. Figure 3 and Table 1 give some data relative to power developments and storage on the Tennessee River and its tributaries.

In any study concerning the advisability of storage, consideration should be given to the possible required use of storage for purposes other than power regulation. For instance, regulation for the Tennessee Valley Authority Dams is complicated by considerations of navigation and flood control. In many places the law or public bodies or courts acting in accordance with law require

TABLE 3

MONTHLY DISCHARGE OF TYPICAL RIVER FOR THE YEAR ENDING DEC. 31, 1941
Drainage Area, 2000 Square Miles

Month	Discharge in Second-feet				Runoff		Accuracy
	Maximum	Minimum	Mean	Mean per Square Mile	Depth in Inches on Drainage Area	Total in Acre-feet	
January	4,320	3,200	3,820	1.91	2.20	235,000	B
February	12,400	3,850	6,650	3.32	3.46	370,000	B
March	12,800	5,900	10,500	5.25	6.06	647,000	B
April	10,300	5,000	7,710	3.85	4.30	459,000	A
May	8,700	1,000	3,850	1.93	2.22	237,000	A
June	1,600	600	860	0.43	0.48	51,200	A
July	1,370	650	1,040	0.52	0.60	64,100	A
August	4,200	1,040	2,500	1.25	1.44	154,000	A
September	6,400	2,000	4,020	2.01	2.24	239,000	A
October	7,000	1,400	3,300	1.65	1.90	203,000	A
November	2,940	1,710	2,330	1.16	1.30	139,000	A
December	4,850	3,015	3,880	1.94	2.24	239,000	B
The year	12,800	600	4,205	2.10	2.37	253,100	

a certain minimum release from a storage reservoir for water supply, maintenance of fish life, preservation of suitable sanitary conditions, use of existing plants, etc. All such regulations and requirements generally result in either less flow available for power regulation or in a larger and more expensive storage reservoir than would be necessary if the project could be utilized exclusively for power regulation.

8. Tabulating Stream Flow. Tables 2–4 indicate the flow of a typical river, the actual discharges of which have been altered to cover desired conditions and to clarify explanations and examples. Tables 2 and 3 are in the form in which such data are usually presented in the *U. S. Geological Survey Water Supply Papers.* Table 4 is a compilation of the average monthly

TABLE 4

AVERAGE MONTHLY DISCHARGE OF TYPICAL RIVER FOR THE YEARS 1923 TO 1942, INCLUSIVE

Drainage Area, 2000 Square Miles

Year	Jan.	Feb	Mar	Apr.	May	June	July	Aug	Sept	Oct	Nov.	Dec
1923	4,000	6,500	13,100	14,100	9,100	1,210	910	2,250	3,700	4,000	2,500	4,000
1924	7,400	6,400	18,700	19,700	5,000	1,380	1,360	2,100	3,900	4,700	2,900	4,300
1925	4,300	7,000	14,500	18,800	4,200	3,050	1,420	3,200	4,600	6,500	5,900	6,500
1926	7,200	3,900	10,400	17,900	8,800	4,300	3,500	3,400	6,500	6,800	6,500	7,700
1927	8,500	4,500	16,200	15,200	3,100	1,190	3,330	5,000	4,700	5,000	6,900	7,100
1928	8,600	7,200	24,000	18,000	7,200	3,020	1,020	2,450	4,100	3,600	2,300	4,000
1929	7,600	9,200	22,000	10,200	7,900	2,500	1,580	2,500	4,000	7,100	3,300	7,400
1930	8,400	6,500	15,300	16,300	6,200	970	2,710	3,000	4,500	7,400	5,500	4,900
1931	6,800	8,700	11,100	8,800	2,700	1,070	2,400	3,500	4,800	5,300	7,200	1,700
1932	6,500	6,300	16,200	17,900	5,600	1,430	990	2,050	5,000	3,500	2,400	5,000
1933	4,600	6,300	8,800	14,400	6,300	1,510	1,630	2,600	6,000	3,800	7,500	8,200
1934	8,200	7,300	10,400	13,100	3,800	2,130	1,750	4,500	4,800	7,700	3,700	6,700
1935	6,200	4,300	12,600	16,100	4,800	1,010	1,460	3,700	4,400	5,600	7,900	3,500
1936	8,000	7,900	18,300	19,500	6,500	920	1,710	2,900	4,100	1,100	5,200	5,200
1937	5,900	3,700	14,500	12,700	5,100	3,500	1,370	2,700	4,900	7,900	4,100	5,600
1938	4,900	6,200	20,700	14,300	10,700	850	1,240	2,100	3,600	5,000	2,500	4,500
1939	7,800	7,500	16,600	20,600	7,400	1,630	1,480	3,900	4,900	5,900	8,100	3,000
1940	5,500	8,100	9,800	14,500	8,500	2,750	1,670	4,100	1,200	1,400	4,500	5,000
1941	3,820	6,050	10,500	7,710	3,850	860	1,040	2,500	4,020	3,300	2,330	3,880
1942	5,200	6,000	12,100	10,600	9,000	1,860	1,550	2,800	4,300	6,200	4,900	5,900

flows for the years of record and is in the form frequently used in engineering reports.

Tabulations are sometimes more conveniently made in second-feet per square mile of drainage area.

The "typical river" will be used in all the examples that follow.

9. The Hydrograph. A hydrograph is a graphical indication of the rate of flow of a stream during a given period, the discharge being plotted as ordinates and time as abscissas. Figures 4 and 5 are, respectively, typical hydrographs of a very flashy stream and of a stream having unusually steady

flow. Figure 6 is a minimum year daily hydrograph of the typical river, plotted from Table 2, and Fig. 7 is a monthly hydrograph of the same year, plotted from Table 3.

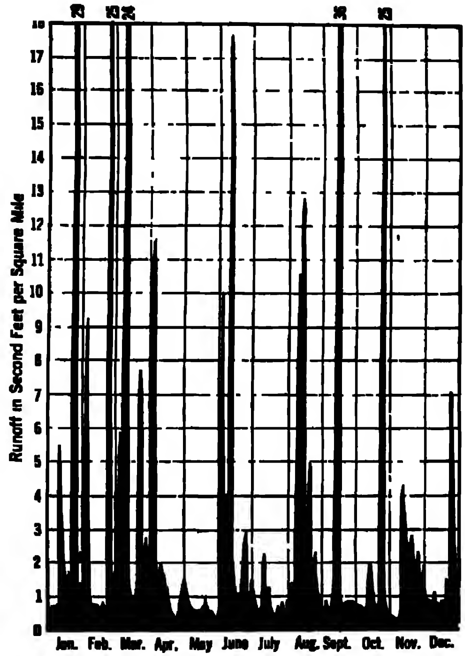

FIG. 4. Hydrograph of Perkiomen Creek near Frederick, Pa., 1904.

In Fig. 6 a flow demand of 3000 sec-ft has been indicated. This demand corresponds to Case 1a of Section 2. If all flows above the flow demand of 3000 sec-ft are discarded, the total area below the demand line and the hydro-

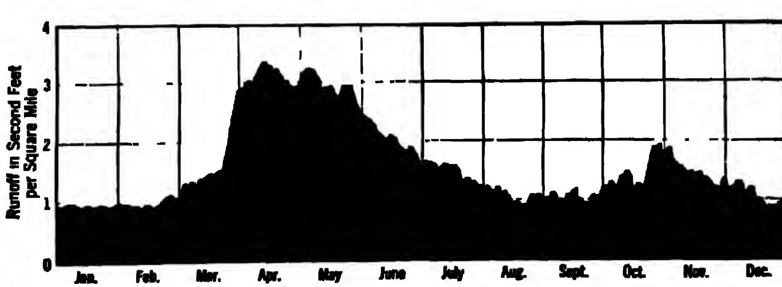

FIG. 5. Hydrograph of the Richelieu River at Fort Montgomery, N. Y., 1904.

graph, or 29,200 sec-ft mo, is the total natural flow available for power or 21,300,000 sec-ft hr during that year. From Eq. 12 of Section 3, Chapter 9, this corresponds to 41,700,000 kw-hr under a productive head of 33 ft and

70% efficiency. Similarly, the total of the shaded areas, or 6800 sec-ft mo, equivalent to 9,720,000 kw-hr, is required from storage or auxiliary plant, as the case may be.

In this example the flow demand is constant throughout the year; but, in many instances, requirements are such that the demand in certain seasons is greater than in others. This may be caused by fluctuating market requirements, variations in head due to draft for storage, or other circumstances, as will be shown later.

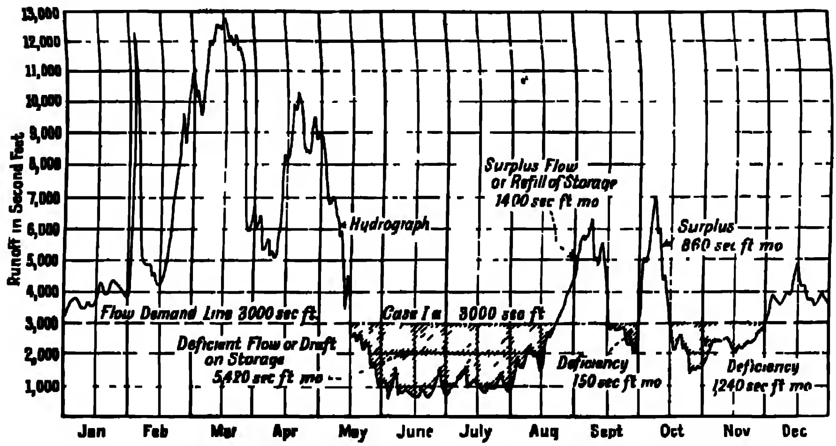

FIG. 6. Hydrograph of typical river showing storage requirements.

With a full storage reservoir at the plant on May 15, the deficient flow from that date to Aug. 22 would require a draft on storage of 5420 sec-ft mo to provide the flow demand of 3000 sec-ft. Between Aug. 22 and Sept. 15, the excess of flow of 1400 sec-ft mo would reduce the reservoir depletion from 5420 to 4020 sec-ft mo. On Oct. 1 the depletion would be 4020 + 150 = 4170 sec-ft mo; on Oct 15, 4170 − 860 = 3310 sec-ft mo; and on Dec. 1, 3310 + 1240 = 4550 sec-ft mo.

It is therefore seen that, on Aug. 22, the reservoir would have been drawn to its fullest extent, or 5420 sec-ft mo, and this represents the capacity required to regulate the flow to 3000 sec-ft for the year 1941. If, between Dec. 1 and the following period of deficient flow, there are 4550 sec-ft mo of excess flow above the 3000 sec-ft demand line, the reservoir will again become full in preparation for service during the next dry season.

Figure 7 is a monthly hydrograph of the typical river for the same period covered by Fig. 6. The use of monthly hydrographs to determine storage requirements over short periods is likely to lead to considerable error, as will be noted by a comparison of the shaded areas of Figs. 6 and 7 indicating required storage capacities of 5420 and 4600 sec-ft mo respectively. Figure 7 fails to show the deficient flows of May, September, and October, and the deficiency in August is also incorrectly indicated. It will be observed that,

when the monthly flow is entirely below or above the demand line, the average monthly flow can be used without error.

For Case 1*b*, with neither pondage nor storage, the stream flow is reduced to usable flow and plotted as shown by the dotted line in Fig 8. The shaded

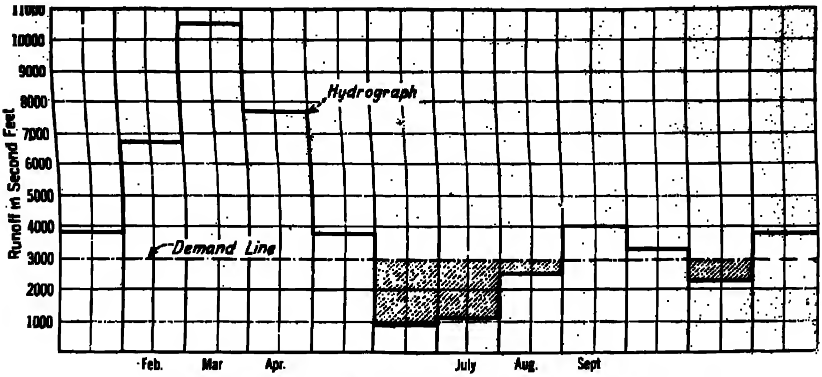

FIG. 7. Monthly hydrograph of typical river for the year 1941.

area then represents the reduction in output due to lack of pondage. To provide full output at all times, storage must be provided to regulate the flow to 4620 sec-ft as that corresponds to the peak demand. Storage re-

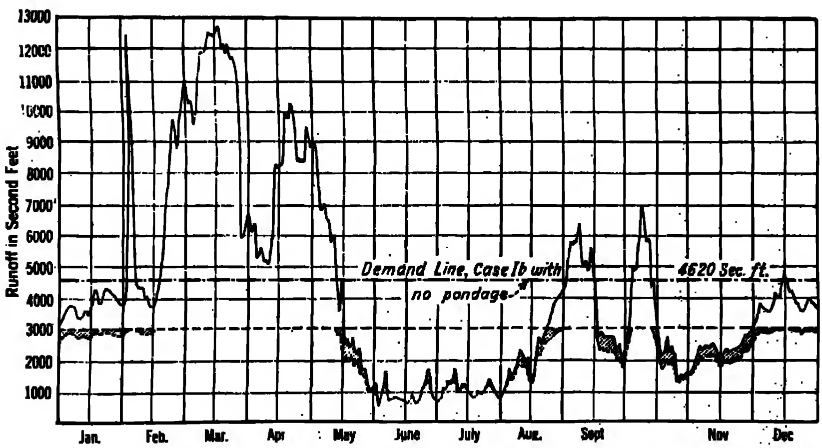

FIG. 8. Daily hydrograph of typical river showing storage requirements with no pondage at plant.

quirements are then computed to the 4620 sec-ft demand line instead of to the 3000 sec-ft line of the previous example. Thus it is seen, that, in this case, the lack of a comparatively small pond at the powerhouse makes it necessary to provide about double the storage capacity for continuous output.

As a representative example of extreme nonuniform flow demand, let Fig. 6 represent a hydrograph of the flow from the area intermediate between the development and a distant storage reservoir. Let Fig. 9 be the hydrograph of the runoff from the area above the reservoir. The runoff from the intermediate area is sufficient for all requirements except during the periods indicated by the shaded area in Fig. 6, and this deficiency of flow is laid off as the variable flow demand in Fig. 9. The total hatched area of Fig. 9, therefore, indicates the flow required from the area above the reservoir, and the double-hatched area the deficiency, or storage requirements.

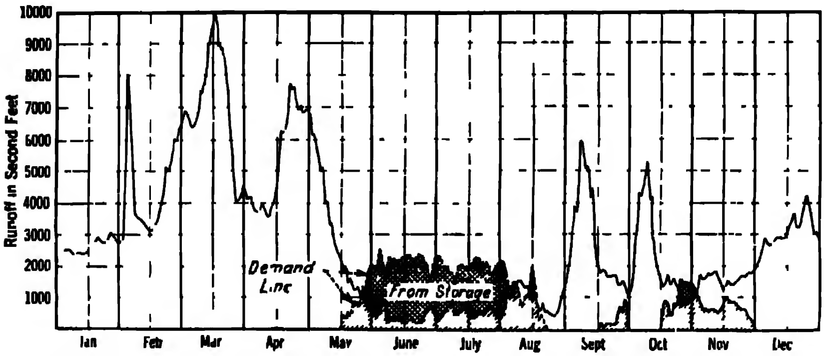

FIG. 9. Hydrograph for portion of typical river above storage reservoir site

A reservoir can be operated to serve to best advantage only one point on the stream. In the last example it will be noted that a development immediately below the reservoir would be provided with a discharge of an extremely fluctuating nature and entirely unsuited to continuous output. If a reservoir is to serve several developments on the same stream, a compromise operation must be adopted to the end that the greatest benefit will accrue to the system as a whole.

High-head plants on small drainage areas are sometimes admirably suited to supplement low-head plants on large streams having no storage facilities. The hydrograph indicated in Fig. 6 is for a 33-ft-head development at a site having a watershed of 2000 sq mi.

Figure 10 shows a hydrograph of an adjoining stream draining only 40 square miles but having an available head of 660 ft. One second-foot under the larger head will produce as much power as $^{660}/_{33}$ = 20 sec-ft on the smaller head. Therefore, at the high-head plant, one twentieth of a second-foot must be utilized for each second-foot of deficient flow at the low-head plant; and, to supplement the total deficient flow of 6810 sec-ft mo at the low-head plant, only one twentieth of this amount, or 340 sec-ft mo, must be converted into power under the higher head, properly distributed as indicated in Fig. 10. The required storage is 215 sec-ft mo.

The feasibility of this arrangement depends upon the cost of installing and operating a high-head plant of nearly the capacity of the low-head plant, with a 215 sec-ft mo reservoir, as compared with the installation and operation of a 5420 sec-ft mo reservoir to regulate the flow at the low-head plant or the installation and operation of an alternative fuel-burning power plant.

If, in the preceding example, the two plants are on the same stream, two revisions must be made in the foregoing calculations. First, the estimated deficiency of flow at the lower plant must be calculated for the area intermediate between the two plants, or 1960 sq mi. Second, since the water re-

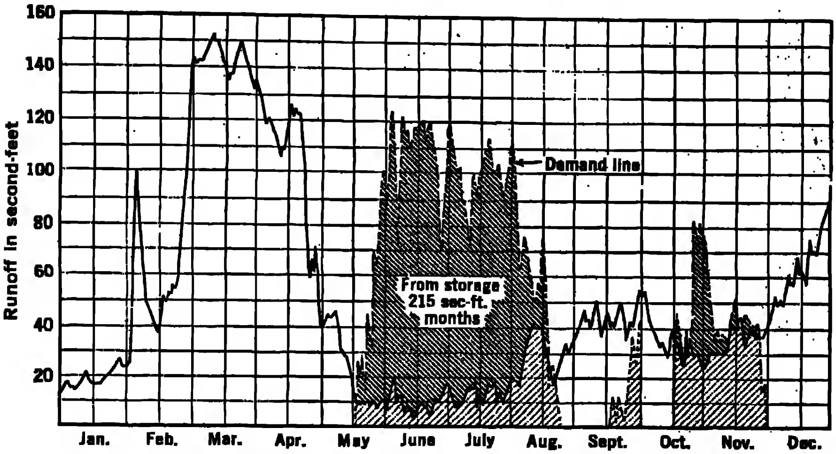

FIG. 10. Hydrograph for small watershed adjacent to typical river.

leased from the upper area, is usable also at the lower plant, it acts under a total head of 693 ft. Therefore, at the high-head plant, $^{33}/_{693} = 1/21$, instead of $1/20$, of the low-head plant deficient flow from 1960 sq mi must be released from above the site of the high-head plant and converted into power under the total head of 693 ft.

The hydrograph is perhaps the most easily understood method of indicating diagrammatically the flow of a stream and the regulation of the flow by storage, particularly for those who are not engineers. It is used extensively for explanatory diagrams; but for estimates of power and storage, other methods often involve less work.

10. The Mass Curve. Mass curves are used only to facilitate storage computations. They indicate the total volume of runoff in second-feet months, or other convenient equivalent, during a given period. They are usually made from records of average monthly flows, these records being summed up consecutively and each sum plotted above the corresponding date.

A mass curve of the typical river for the year 1941–1942 is indicated by the heavy line of Fig. 11. The computations for this curve are given in Table 5.

This mass curve indicates that, during the period from Apr 1, 1941, to Mar 31, 1942, 52,709 sec-ft mo passed the site

It is evident that a mass curve plotted from mean monthly flows is correct only at the beginning and end of each month, since the variation in flow during

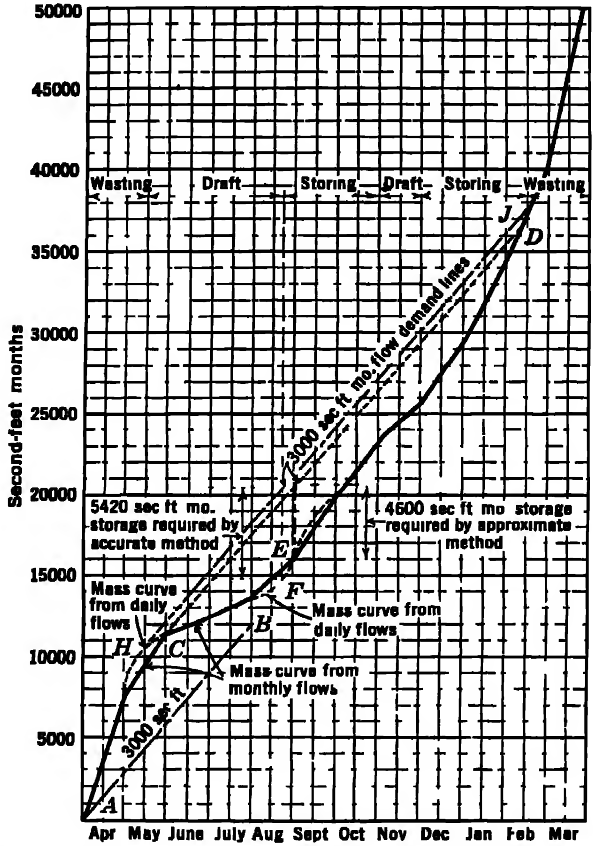

FIG 11 Mass curve of typical river for the year 1941–1942 the lowest of record plotted from Table 5

the month is not taken into consideration For a uniform monthly flow, the mass curve for the month would be a straight line A true curve for any month would be convex upwards if the flow were greater during the first part of the month A summation of daily flows, instead of monthly flows, results in a more accurate mass curve, but this involves an excessive amount of work, and daily flows are seldom used except for critical periods as explained later

The slope of the curve at any point indicates the rate of flow in second-feet If the curve is horizontal, the flow is zero. The slope of a line connecting

TABLE 5

COMPUTATIONS FOR MASS CURVE OF TYPICAL RIVER FOR THE YEAR 1941–1942

Month	Second-feet, Mean Monthly Runoff	Second-feet, Months Accumulative	Month	Second-feet, Mean Monthly Runoff	Second-feet, Months Accumulative
April	7,710	7,710	October	3,300	23,280
May	3,850	11,560	November	2,330	25,610
June	860	12,420	December	3,880	29,490
July	1,040	13,460	January	5,200	34,690
August	2,500	15,960	February	6,000	40,690
September	4,020	19,980	March	12,100	52,790

any two points, *CD*, on the curve is a measure of the average flow during that period, the actual flow being less during the first and greater during the last part of the period. At point *C* the accumulative flow is 11,560, and at *D* 36,700 sec-ft mo. The difference, or 25,140, is the total flow for the period from *C* to *D*, or for 8.38 months. The average flow is therefore 25,140/8.38 = 3000 sec-ft. The average flow can be obtained more easily by drawing a line, *AB*, through the origin of the curve, parallel to *CD*. At the end of the first month the ordinate to the line *AB* is 3000 sec-ft.

The 3000 sec-ft flow demand of Fig. 11 will be used to explain the determination of storage requirements from the mass curve. As the mass curves for April and May are both steeper than the 3000 sec-ft line, *AB*, the flow during those months is indicated to be in excess of the demand, and no storage is required. The curve for June, however, has a flatter slope. From the point *C*, draw the line *CD* parallel to *AB* representing the demanded flow of 3000 sec-ft. With a full reservoir at *C*, draft will occur at all times when the slope of the mass curve is less than the line *CD*, and the reservoir will be filling when the slope is greater. The depletion of storage at any time is measured by the ordinates between the curve and the line, the reservoir being full at both points *C* and *D*. The greatest draft from storage, as measured by the length of the maximum intercepted ordinate at *E*, is 4600 sec-ft mo.

Monthly mass curves, like monthly hydrographs, are subject to errors because of their representing average rather than actual monthly flows. The errors occur at the beginning and at the end of reservoir drawdown. The dotted lines in Fig. 11 indicate the plotting of daily flows for the months of May, August, and September. It will be noticed that the slope of the correct mass curve during the latter part of May is less than the demand line, beginning with the 15th of the month, or at *H*, where a new demand line would be

tangent to the corrected curve. The maximum ordinate is now at *F* and shows the correct greatest draft to be 5420 sec-ft mo.

If the flow-demand line does not intersect the mass curve, as at *J*, the reservoir will not be filled again. If the reservoir is very large, the time interval between the points *H* and *J* may be several years.

To determine whether the reservoir would be full at *H*, a mass curve must be drawn for the preceding year and investigated in the same manner.

The flow-demand line may be curved if the seasonal power demand is not constant. In the preceding example, the storage is at the plant. For remote storage the use of the mass curve is not as adaptable as other methods.

In studies involving the regulation of stream flow over periods in excess of one year, it is desirable to plot the mass curve covering the entire period of record. This is particularly true in the case of storage for water supply and/or irrigation purposes where storage capacity may equal the average flow of the stream for several years. In all cases it is necessary to adjust the regulated flow for the evaporation which takes place in the reservoir (see Chapter 2).

11. The Flow Duration Curve. Figures 12 and 13 show flow duration curves for the typical river for the twenty years of record and for the minimum year of record, respectively.

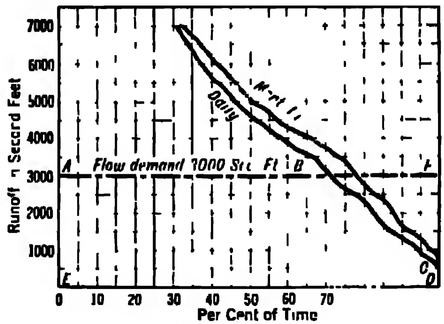

FIG. 12. Monthly and daily flow duration curves of typical river for the 20 years of record

Each point on a flow duration curve indicates the percentage of time during the period under consideration that the flow was equal to or greater than the given discharge. Flow duration curves may be computed from mean monthly flows or daily flows. Those computed from mean monthly flows are only approximate, because in each month there is a considerable variation in flow not indicated in the duration curve. Variations in flow during 24 hours are usually negligible. Figure 12 shows the error involved in the use of mean monthly flows for the duration curve of the typical river.

Computations involving the use of daily flows require the consideration of thirty times as many items as those made for mean monthly flows. The time

consumed in the use of daily flows has led many engineers to use monthly duration curves exclusively, probably without a realization of the errors involved. Such errors will usually range from 5 to 15%, depending upon the characteristics of the stream and the extent of utilization of the flow. The typical river is an average stream. The difference between the daily and monthly curves would probably be negligible for steady streams similar to the Richelieu River indicated in Fig. 5 but greater than that shown in Fig. 12 for very flashy streams.*

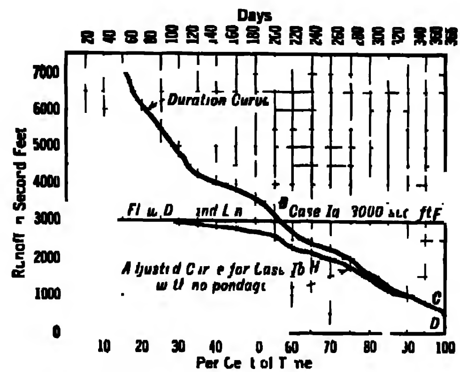

Fig. 13. Daily flow duration curve of typical river for year 1941, the lowest of record, from Table 6.

Computations involved in the construction of the flow duration curve of Fig. 13 are given in Table 6. Rates of flow differing by convenient amounts are set up in Col. 1. In Col. 2 is indicated the number of times the daily flow equaled a value between the corresponding flow in Col. 1 and the flow next below. During the year a daily flow between 1000 and 1100 sec-ft occurred 10 times.

A summation of occurrences is tabulated in Col. 3. It shows that a flow of 1000 sec-ft was equaled or exceeded on 329 days during the year. Column 3 is then reduced to per cent of time and recorded in Col. 4. Columns 1 and 4 are used to plot the flow duration curve. For a flow duration curve of a 1-year period, Col. 2 will total 365 days and Cols. 1 and 3 may be used, if desired, to indicate the number of days during the year a given flow was exceeded, as indicated in Fig. 13. For a longer period than a year, Col. 2 will total more than 365 days, but a flow duration curve for the total years of record may be used to represent an average year, and, if days are desired at the bottom of the diagram, percentages are simply multiplied by 365.

Each square in Figs. 12 and 13 is $500 \times 5 \times 365 \div 100 = 9130$ sec-ft days or 300 sec-ft mo.

* For other comparisons of daily and monthly duration curves see *News-Rec.*, Vol. 87, p. 250, and Ninth Annual Report of New York State Conservation Commission, 1919.

TABLE 6

Computations for Daily Duration Curve of Typical River for the Year 1941, the Minimum of Record

Plotted in Fig. 13

(1) Second-feet Runoff	(2) Number of Days	(3) Days Equaled or Exceeded	(4) Per Cent of Time
500	1	365	100
600	12	364	99.7
700	8	352	96.4
800	6	344	94.2
900	9	338	92.6
1,000	10	329	90.1
1,100	6	319	87.4
1,200	7	313	85.7
1,300	3	306	83.8
1,400	9	303	83.0
1,600	13	294	80.5
1,800	6	281	77.0
2,000	14	275	75.3
2,200	21	261	71.5
2,400	14	240	63.7
2,600	10	226	61.9
2,800	10	216	59.2
3,000	5	206	56.4
3,200	5	201	55.0
3,400	18	196	53.7
3,600	11	178	48.7
3,800	17	167	45.7
4,000	13	150	41.1
4,200	13	137	37.5
4,400	10	124	34.0
4,600	2	114	31.2
4,800	8	112	30.7
5,000	13	104	28.5
5,500	13	91	25.0
6,000	14	78	21.4
6,500	4	64	17.5
7,000	5	60	16.5
7,500	0	55	15.1
8,000	6	55	15.1
8,500	10	49	13.4
9,000	3	39	10.7
9,500	5	36	9.9
10,000	8	31	8.5
10,500	4	23	6.3
11,000	1	19	5.2
11,500	5	18	4.9
12,000	9	13	3.6
12,500	4	4	1.1
	365		

A flow demand of 3000 sec-ft or 36,000 sec-ft mo per year has been indicated in Figs. 12 and 13. The total area $ABCDE$, below the demand line and the flow duration curve, or 29,200 sec-ft mo for the minimum year and 31,600 sec-ft mo for the average year, represents the total natural flows available for power during the year. From Eq. 10 of Section 3, Chapter 9, these correspond to 41,700,000 kw-hr and 45,200,000 kw-hr, respectively, under a productive head of 33 ft and efficiency of 70%.

The areas BFC, or 6800 sec-ft mo in the minimum year, and 4400 sec-ft mo in the average year, equivalent to 9,720,000 kw-hr and 6,280,000 kw-hr, respectively, are a measure of the power required from storage or auxiliary plants as the case may be. The curves also show that natural flow sufficient for full output is available 56.5% of the time, or 206 days, in the minimum year, and 71% or 259 days, in the average year.

The area BFC is the total discharge from storage required to supplement the natural flows; but it is no indication of the required capacity of the reservoir, as the reservoir may be filled and emptied more than once during the year. This feature is more clearly indicated in Section 9 which shows that for 1941, the minimum year, a capacity of 5420 sec-ft mo was required to supply the total demand of 6800 sec-ft mo.

For Case *1b*, with neither pondage nor storage, the steam flow, as indicated by the duration curve, is reduced to usable flow and plotted as shown by the line $AGHC$ of Fig. 13. The area $GHCB$ then represents the reduction in output due to lack of pondage, and the area $FBGHC$ the output required from storage or auxiliaries.

For the case of remote storage but no pondage, the regulated flow at the plant may be computed by analytical methods. If the reservoir is not regulated for the plant in question, or if it is of capacity sufficient only for partial regulation, the flow at the plant can be plotted as a duration curve, then adjusted for lack of pondage as previously described, and the output computed.

12. Analytical Methods Applied to Storage Studies. Analytical methods are adaptable only to cases where adequate pondage is available. Average monthly flows must be used exclusively because a consideration of daily flows would result in an endless amount of labor. An example of calculation by analytical methods will be given for a plant having no storage, and one for a plant with storage A constant flow demand of 3000 sec-ft, corresponding to Case *1a* of Section 2, will be used in both examples.

No Storage. If, in any month, the average flow is equal to or greater than the flow demand of 3000 sec-ft, it does not follow that without storage all power demands will be met. In Table 3, the average monthly flow of the typical river during May 1941, is given as 3850 sec-ft. By reference to Table 2, however, it is seen that the flow during part of that month was considerably less than the flow demand of 3000 sec-ft, resulting in a deficiency in output. Moreover, if the average flow is less than the flow demand, it is not necessarily true that, without storage, all the flow can be converted into power, because

it is quite possible that, during some part of the month, the flow was in excess of the flow demand and some waste was necessary. This feature has led to the adoption of *use curves*, samples of which are shown in Fig. 14.

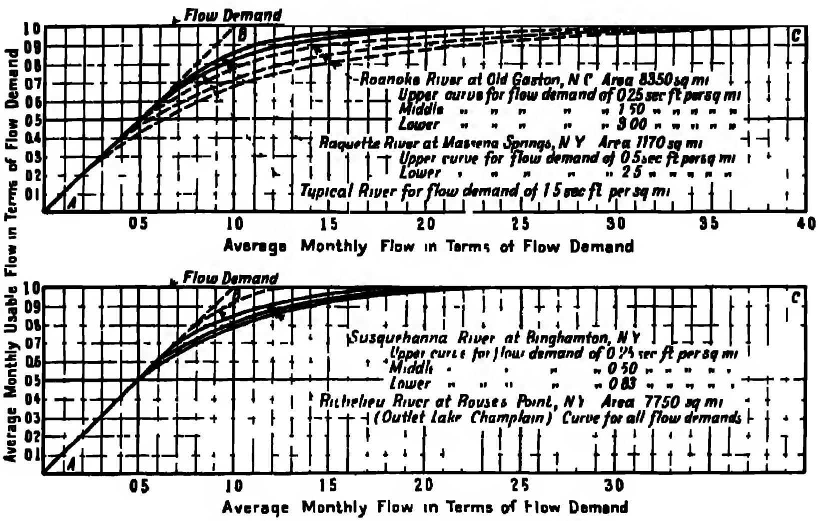

FIG 14. Typical use curves.

Use curves show the relation between average monthly flow and probable usable flow, both expressed either in terms of the flow demand or of second-feet flow. The use curve of the typical river is shown in both ways, the latter

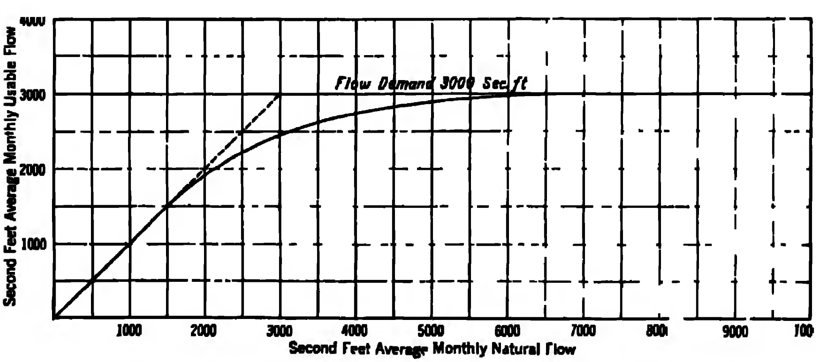

FIG 15 Use curve of typical river.

indicated in Fig. 15. If, for instance, the average flow of the typical river for a given month was 1.2 times the flow demand, or 3600 sec-ft, the probable average usable flow during that month is indicated in Figs. 14 and 15 to be 0.88 times the flow demand or 2640 sec-ft.

The use curves of Fig. 14 were plotted from an analysis of the daily records of a number of years of each stream. It will be noticed that, for very steady streams such as the Richelieu (see Fig. 5), the use curve approaches the line ABC which corresponds to the case where the flow is absolutely constant during each month. The construction of a use curve involves considerable labor, and, for approximate computations, an existing curve of a similar stream may be used without great error.

The average monthly flows shown in Table 4 have been reduced to approximate average monthly usable flows in Table 7 by means of the use curve of Fig. 15. The average usable flow for the twenty-year period is seen to be 2630 sec-ft or 31,560 sec-ft mo per annum. This corresponds to the 31,600 sec-ft mo in the average year found by the flow duration curve method of Section 11.

TABLE 7

COMPUTATIONS FOR AVERAGE USABLE FLOW, TYPICAL RIVER

Flow Demand, 3000 Sec.-ft. No Storage

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Average
1923	2740	3000	3000	3000	3000	1210	910	2070	2670	2740	2210	2740	2480
1924	3000	3000	3000	3000	2900	1380	1360	2160	2730	2850	2420	2800	2550
1925	2800	3000	3000	3000	2780	2470	1420	2520	2840	3000	2900	3000	2780
1926	3000	2720	3000	3000	3000	2800	2630	2590	3000	3000	3000	3000	2800
1927	3000	2820	3000	3000	2500	1190	2570	2900	2850	2900	3000	3000	2720
1928	3000	3000	3000	3000	3000	2470	1020	2200	2760	2640	2100	2740	2580
1929	3000	3000	3000	3000	3000	2220	1580	2200	2740	3000	2500	3000	2600
1930	3000	3000	3000	3000	3000	970	2310	2450	2830	3000	2930	2880	2700
1931	3000	3000	3000	3000	2320	1070	2100	2610	2870	2930	3000	2850	2050
1932	3000	3000	3000	3000	2940	1430	900	1950	2900	2620	2160	2900	2490
1933	2840	3000	3000	3000	3000	1510	1600	1970	2970	2700	3000	3000	2630
1934	3000	3000	3000	3000	2700	2000	1700	2820	2870	3000	2670	3000	2730
1935	2980	2800	3000	3000	2800	1040	1460	2670	2820	2940	3000	2620	2590
1936	3000	3000	3000	3000	3000	920	1700	2420	2760	2760	2920	2950	2620
1937	2970	2070	3000	3000	2930	2620	1370	2320	2880	3000	2700	2940	2700
1938	2880	3000	3000	3000	3000	850	1210	1900	2050	2900	2220	2820	2460
1939	3000	3000	3000	3000	3000	1630	1480	2720	2880	2970	3000	2450	2680
1940	2940	3000	3000	3000	3000	2350	1630	2760	2780	2810	2830	2940	2750
1941	2700	3000	3000	3000	2700	860	1040	2200	2750	2550	2120	2720	2390
1942	2930	2970	3000	3000	3000	1800	1540	2380	2800	3000	2880	2970	2600
										General average			2630

From Table 7, the average usable flow for the minimum year, 1941, is 2390 sec-ft or 28,680 sec-ft mo, corresponding to the 29,200 sec-ft mo found by the hydrograph and duration curve methods for that year.

The deficiency, to be made up from some other source, is the difference between the second-feet months of usable flow and the flow demand of 36,000 sec-ft mo per annum, or 4440 sec-ft mo in the average year, and 7320 sec-ft

mo in the minimum year. The usable flows and deficiencies may be converted into units of energy by Eq. 10 of Section 3, Chapter 9, as previously explained.

With Storage at the Plant. With storage at the plant, the problem is complicated by the fact that the flow demand is not constant, because of the necessary reduction in head during draft. For simplicity, however, a constant flow demand of 3000 sec-ft will be used in the following example as in the preceding one.

Table 8 indicates the usual approximate analytical method for determining storage requirements when the reservoir is at the plant. The method

TABLE 8

TYPICAL RIVER, YEARS 1941 AND 1942, APPROXIMATE COMPUTATIONS FOR STORAGE REQUIREMENTS, RESERVOIR AT PLANT

Second-feet Months

	Jan	Feb	Mar.	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
1941												
Natural stream flow					3 850	860	1 040	2 500	4 020	3 300	2 330	3 880
Draft from storage					0	2 140	1 960	500			670	
Storage refill									1 020	300		880
Regulated stream flow					3 850	3 000	3 000	3 000	3 000	3 000	3 000	3 000
Reservoir depletion					0	2 140	4 100	4 600	3 580	3 280	3 950	3 070
1942												
Natural stream flow	5 200	6 000	12 000	10 600	9 600	1 860	1 530	2 800	4 300	6 200	4 900	
Draft from storage			0	0	0	1 140	1 450	200			0	
Storage refill	2 200	870							1 300	1,490		
Regulated stream flow	3,000	5 130	12 100	10 600	9 600	3 000	3 000	3 000	3 000	4 710	4 900	
Reservoir depletion	870	0	0	0	0	1 140	2 590	2 790	1 490	0	0	

is obvious. It shows the greatest draft on storage for the typical river in 1941 to be 4600 sec-ft mo

This corresponds to the use of the monthly hydrograph of Fig. 7 for computing storage requirements and involves the error, as explained in Section 9, that the latter part of May and the first part of September are deficient in flow, as indicated in Fig. 6, notwithstanding the fact that the average monthly flows of these months are considerably above the flow demand of 3000 sec-ft. Such errors become smaller as the period of deficient flow becomes longer, and for relatively large reservoirs, depleted for long periods, the error will be negligible. For short periods of depletion, those months in which the flow is partly below and partly above the flow demand must be examined in detail.

With Storage Remote from Plant. If the storage is remote from the plant, Table 9 indicates the usual method of calculating storage requirements.

TABLE 9

TYPICAL RIVER, YEARS 1941 AND 1942, APPROXIMATE COMPUTATIONS FOR STORAGE REQUIREMENTS, RESERVOIR REMOTE FROM PLANT

Second-feet Months

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1941												
Natural flow below reservoir					3,465	774	936	2,250	3,618	2,970	2,097	3,492
Flow past reservoir					0	2,226	2,064	750	0	30	903	0
Total flow at plant					3,465	3,000	3,000	3 000	3,618	3,000	3,000	3,492
Natural flow above reservoir					385	86	104	250	402	330	233	388
Draft from storage					0	2,140	1 960	500			670	. ..
Storage refill					0				402	300		388
Reservoir depletion		.			0	2,140	4 100	4,600	4 198	3 898	4,568	4,180
1942												
Natural flow below reservoir	4 680	5 400	10 890	9 540	8 640	1,674	1 395	2,520	3,870	5,580	4,410	5,310
Flow past reservoir	0	0	0	0	170	1,326	1 605	480	0	0	0	0
Total flow at plant	4 680	5 400	10 890	9,540	8 810	3,000	3,000	3 000	3,870	5 580	4,410	5,310
Natural flow above reservoir	520	600	1,210	1 060	960	186	155	280	430	620	490	590
Draft from storage						1 140	1 450	200				
Storage refill	520	600	1 210	1,060	790				430	620	490	590
Reservoir depletion	3 660	3 060	1 850	790	0	1,140	2 590	2,790	2,360	1,740	1,250	660

In this example it has been assumed that one tenth of the flow of the previous example is available from the area below the reservoir, and nine tenths from the area above. These are indicated in Lines 1 and 4. The flow past the reservoir, when the reservoir is partly depleted, must be such that the total flow at the plant equals the flow demand, and the draft on storage or refill is the algebraic difference between Lines 4 and 2.

It will be noticed that, here, the 1941 draft on storage is greater than for the previous example, because, in this case, an excess flow of 618 sec-ft mo from the lower area was wasted past the site in September when the reservoir was in need of replenishment.

13. Functions of Storage in Combined Systems. Most modern power systems are served by both steam and hydro plants. During periods of low stream flow, the hydro plants with pondage serve the peak loads and steam plants the base load, but during periods of high stream flow, when the maximum flow demand (Sect. 3) is equal to or less than the stream flow, the hydro plants operate on the base of the load. In such power systems the most important function of storage is to increase the firm capacity (see Section 3, Chapter 15) of the hydro in present or prospective plants and thus avoid the necessity for installing the same amount of additional steam capac-

ity. Thus, in Fig. 6, Chapter 14, it is evident that if, under minimum flow conditions, only 200,000 kw-hr of energy were available during the peak load week for the hydro plant at the top of the diagram, only a part of the 38,500 kw of installed capacity would be firm. If, however, sufficient storage were added to bring the energy up to 438,000 kw-hr, then the entire installation of 38,500 kw would be firm capacity and perform the same function as alternative steam capacity.

Such storage could confer a similar benefit on other developments, when constructed, on the same river, but it would not be developed unless the increased revenue from the additional firm capacity and accompanying energy

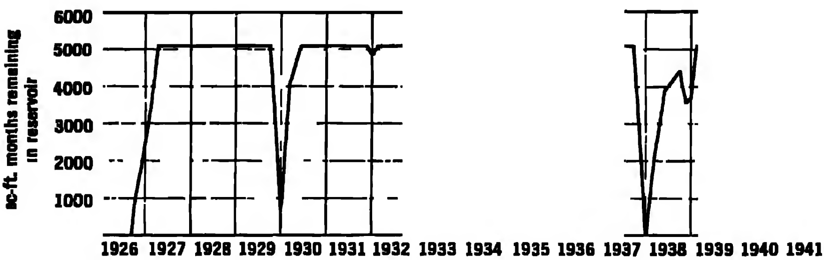

FIG. 16. Gamma River storage possibilities Fluctuations in stage of Delta Reservoir from September 1926 to September 1941 when discharge is regulated to produce the greatest practicable minimum firm capacity at downstream power plants with storage available in this reservoir. (Note: Drainage area of Gamma River at site = 1012 sq mi. Capacity of Delta Reservoir 26.6 billion cu ft 10,130 sec-ft mo. Maximum release from reservoir 5690 sec-ft average weekly flow.)

at the various plants would carry the annual charges both on the additional installation and on the investment in storage plus a profit.

In Fig. 16 is shown a diagram giving the fluctuations in reservoir content over a term of years where the release of water from the reservoir was for the sole objective of maintaining the maximum firm hydro capacity in downriver hydro plants having a total head of 600 ft. It will be noted that most of the time the reservoir is full and that only rarely is its full capacity utilized.

Such reservoirs so operated have some additional insurance value in that, in the event of a steam plant breakdown during low stream flow conditions, they would permit the operation of full hydro capacity with water supplied from the storage reservoir for long periods until repairs were made to the steam plant. After the emergency had passed, the use of stored water could be discontinued, the reservoir allowed to fill, and reserve steam capacity operated over the peaks when required.

Briefly, the most productive method of operating such storage reservoirs is to keep them full as in Fig. 16 except when drafts are required to maintain the firm capacity of the hydro plants or for some other emergency. Frequently there is a power plant at the outlet of the storage reservoir, and a

slight departure from the above principle is desirable. Thus, heavy rains may give warning of an approaching flood on the watershed tributary to the storage reservoir. Under these circumstances, the water surface should be drawn down sufficiently, if possible (the discharge being taken through the power plant), to take the flood in the reservoir without discharge of water over the spillway. With a reservoir as large in relation to watershed area as the Delta Storage reservoir there should be very little wastage over the spillway, and practically all the water from the storage reservoir should pass through all the power plants downstream.

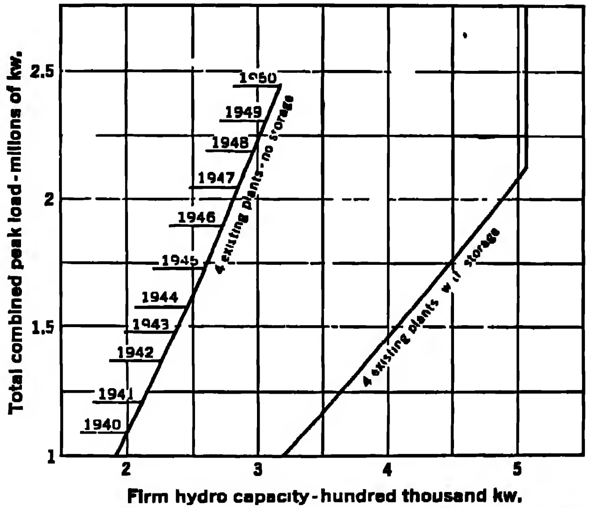

FIG. 17. Effect of storage on firm hydro capacity of Gamma power system.

In Fig. 17 is shown the effect of adding storage on the firm hydro capacity of a large power system all of whose hydro plants are located on the same river. The proposed storage is on a large tributary upstream from the existing power plants, and a power plant is contemplated at the outlet to the storage reservoir.

The curve at the left of the figure shows the theoretical firm hydro capacity of the existing hydro plants with no storage provided. It is plotted on the assumption that the load factor now remains the same as the connected load grows (Section 2, Chapter 14). The load factor is not always static, however, and there is a distinct tendency in many power systems for the load factor to increase as the total load grows (Section 5, Chapter 15). This means that a given amount of capacity serving the peak load will, with an increased load factor, require more hydro energy. Consequently, an increase in load factor enhances the importance of storage.

The curve at the right of the figure shows the firm capacity obtainable in existing plants (provided the necessary capacity is installed) and in the

proposed power plant at the storage reservoir. It will be noted that the increase in firm capacity due to storage exceeds 50%. It is occasionally assumed that storage is merely of temporary advantage and that the same advantage could be obtained by waiting a few years and letting the load curve grow until the proposed installation would be firm capacity. (See curve at the left of Fig. 17.) This is an invalid assumption, however, as the addition of storage for any practical condition of present or future load curves will always permit the installation of additional firm capacity either in the same plant, in another plant, or in a projected plant on the same river or even on another stream.

Whether or not the proposed addition of any available storage capacity is economically advisable in any power supply system depends on careful consideration of the factors discussed in Chapters 12, 13, 14, and 15.

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CHAPTER 11

TYPES OF HYDRO PLANTS

1. Introduction. Hydro plants may be classified on a functional basis as (1) run-of-river plants without pondage, (2) base-load plants, (3) run-of-river plants with pondage, and (4) peak-load plants. Pumped storage plants are simply a special kind of peak load plant.

2. Run-of-river Plants without Pondage. Some plants are located so that they must use the water just as it comes to them without being able to pond it or store it to apply on the peak of the load. Such plants are generally referred to as run-of-river plants without pondage. A usual reason for their lack of pondage is that the dam is constructed to maintain a given water level, as for navigation, and the power plant is only incidental.

The heads are sometimes quite low, and often, at times of flood, tailwater rises to such an extent that the plants are inoperative. A high-head plant may also belong to this class, as one with a dam too low to give pondage at the head of a falls. Although it cannot be said that such plants uniformly have no capacity value (see Section 9, Chapter 15), their capacity is seldom as useful as that of other types of plants. Their principal function is as energy producers to save coal that it would otherwise be necessary to burn in steam plants. The hydro plant of the Louisville Gas and Electric Company (72,000 kw installation) on the Ohio River, and the Winfield, Marmet, and London plants on the Kanawha River are examples of this type.

3. Base-load Hydro Plants. As indicated by the name, these are run-of-river plants without pondage that are capable of substantially continuous operation in the base of the load curve throughout the year. Consequently, their full plant discharge is seldom very much greater than the minimum flow of the river. Examples are Schoelkoff (Fig. 1) and Queenston-Chippewa (Fig. 2) on the Niagara River, Cedar Rapids, and Beauharnhois (Fig. 4) on the St. Lawrence. For all the above plants full plant discharge is very much less than the minimum flow of the river. Bonneville (Fig. 5) on the Columbia River is another example. Also, a number of very high-head plants may be put in this class. The annual capacity factor of base-load plants varies from 70 to 100%.

4. Run-of-river Hydro Plants with Pondage. Usefulness is greatly increased if a plant has pondage and if tailwater conditions are such that floods will not "drown out the plant." By pondage is meant sufficient storage at the plant to take care of hour-to-hour fluctuations in load on the plant throughout the period of a week. In the usual run-of-river hydro plants

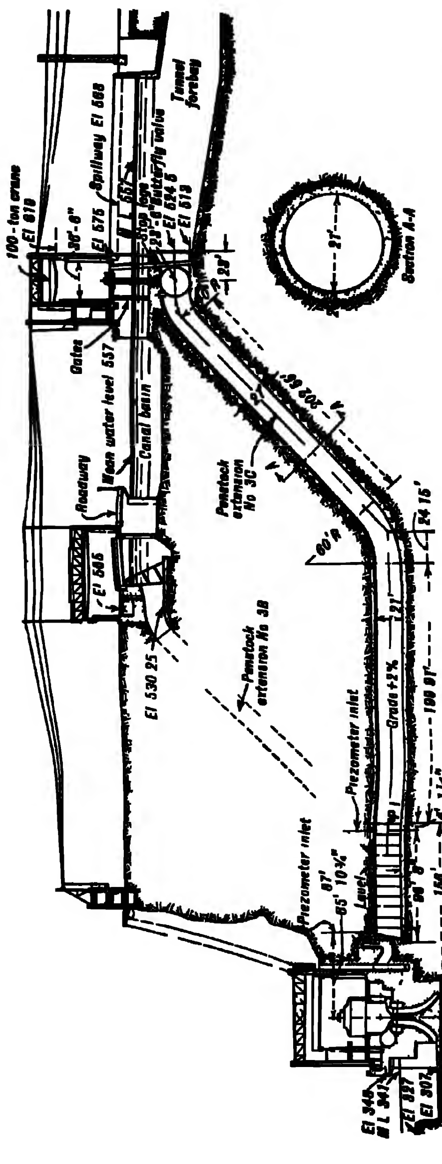

Fig 1. Schoelkoff plant, Niagara Falls Station 3C 452 500 hp 217 ft head (Note Total installation at Niagara Falls for all plants is approximately 1 500 000 hp)

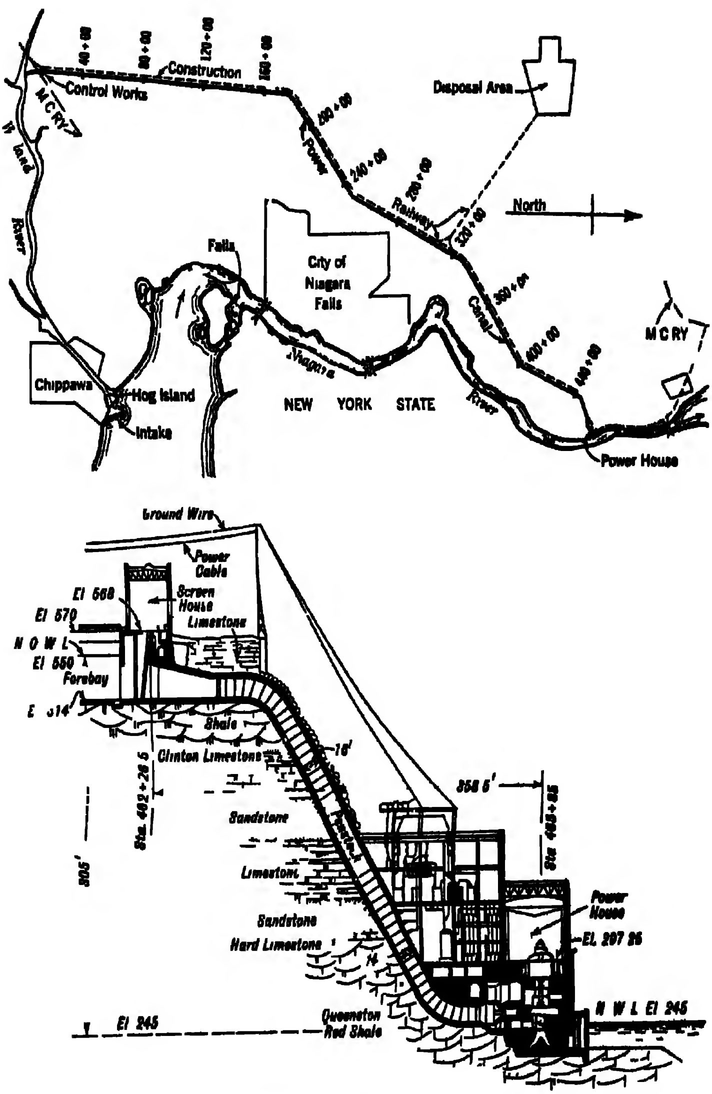

Fig 2 Queenston-Chippawa plant Niagara Falls 500 000 hp 294 ft head

FIG. 3 Niagara Falls the source of a million kilowatts of generated power, 56 000 sec-ft out of a total minimum discharge of 117 000 sec-ft (average = 191 000 sec-ft) is used for the production of power

with pondage, installation is on a high basis in relation to minimum stream flow. In some plants, discharge is 10 to 20 times minimum stream flow. Full plant discharge is often from 1.5 to 3.0 sec-ft per sq mi of watershed area. When connected to loads which are served by both steam and hydro, it is not unusual for practically all the installed capacity to be firm capacity, i.e., capable of performing the same function as alternative steam capacity.

When plenty of water is available in the river, these plants operate on the base of the load curve. With seasonal decrease in stream flow they gradually

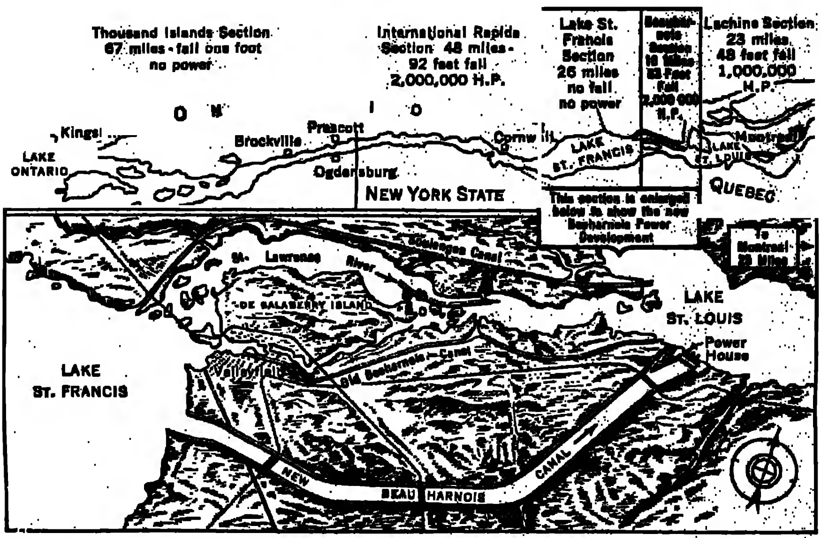

FIG. 4. Beauharnois hydro development on the St. Lawrence River near Montreal, a base load plant, head 83 ft, installation 315,000 kw, 9 units.

change their position on the load curve until under minimum stream-flow conditions they operate entirely on the peaks of the load curve. Figure 6 illustrates how plants of this type alternate between base load and peak load according to the stream flow available.

Annual capacity factors for this type of plant vary from 40 to 65%. Examples that might be mentioned are Fifteen Mile Falls, Mitchell Dam, Conowingo (Fig. 7), and Safe Harbor (Fig. 33, Chapter 38).

5. Reservoir Plants. While hydroelectric projects which take their flow directly or indirectly from large storage reservoirs are suitable for development as peak-load plants, provided that the conduit required is not too long or too expensive, they are not necessarily peak-load plants. This is because market conditions may not require all the peak-load hydro power available at such a site. Consequently, many plants of this type operate on a daily capacity factor (see Section 8, Chapter 12) of 30 to 100%. Notable examples of plants that draw their water supply from large reservoirs are Hoover

FIG. 5. Bonneville development on the Columbia River, Ore., 522,000 kw, 60 ft head, 10 units. Average stream flow 214,000 sec-ft, minimum without regulation 40,000 sec-ft (U. S Engineer Dept)

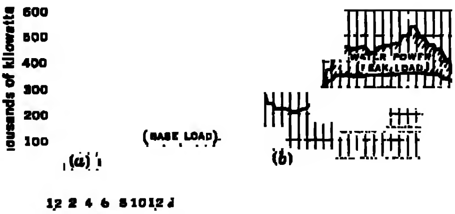

FIG. 6. Utilization of run-of-river plants with pondage Combined 24-hr load curve of Baltimore, Md., and Washington, D. C., showing how systems are served by water power from Holtwood and Safe Harbor plants of Pennsylvania Water and Power Co. (From "Combined Energy Generation," by Ezra B. Whitman, *Trans. A S C E.*, Vol. 104, p. 1120, 1939)

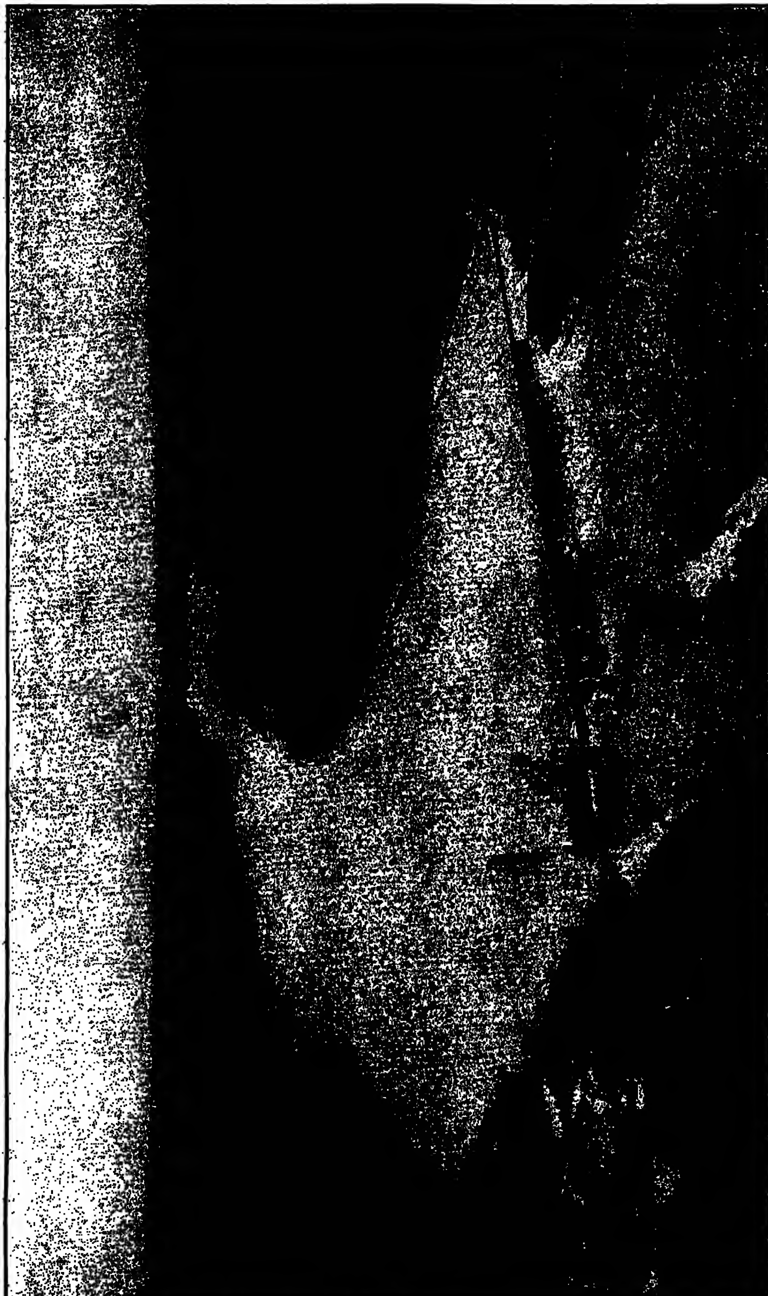

FIG. 7. Conowingo hydro development, Susquehanna River, a run-of-river plant with pondage. Head 89 ft. Installation 252,000 kw, 7 units. Minimum stream flow Susquehanna River 2300 sec-ft (average 39,000 sec-ft), full plant discharge 43,000 sec-ft.

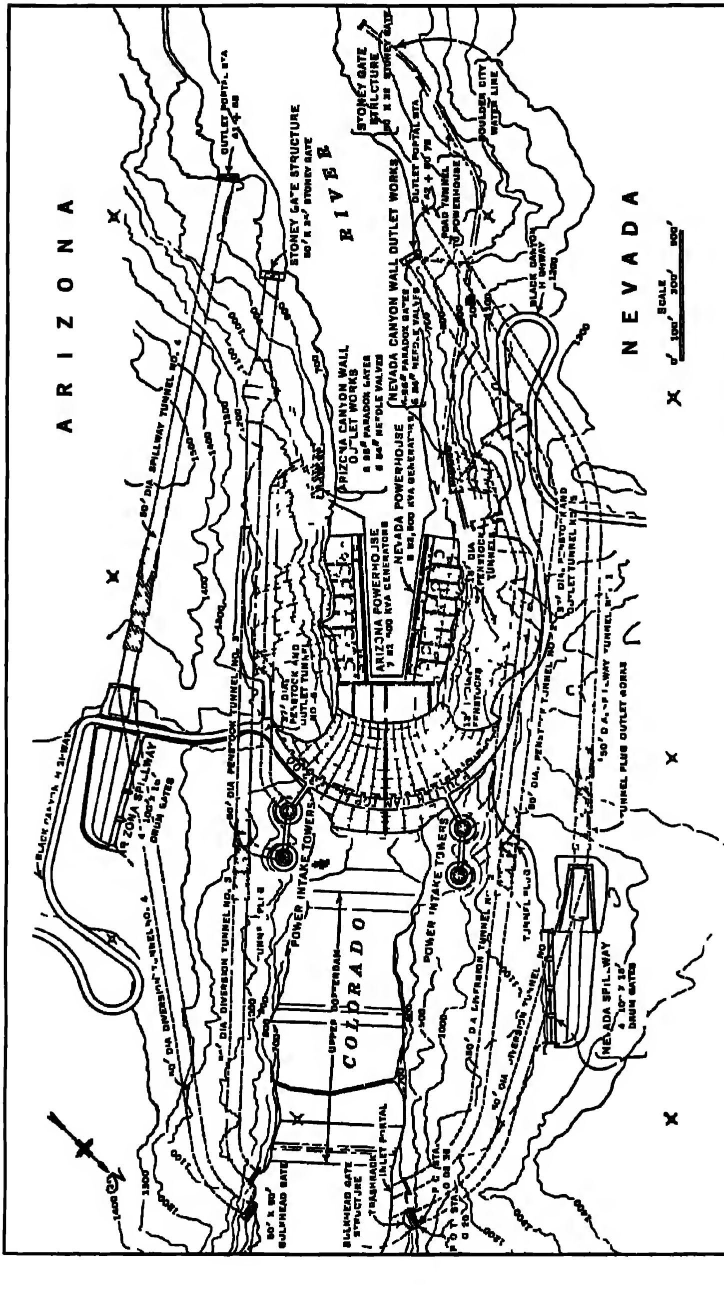

FIG. 87. Hoover Dam development, Colorado River, Nevada-Arizona. 460 000 hp. 1 036 000 kva (1944), 475 ft head. live storage capacity 29 152,000 acre-ft. (U. S. Bureau of Reclamation.)

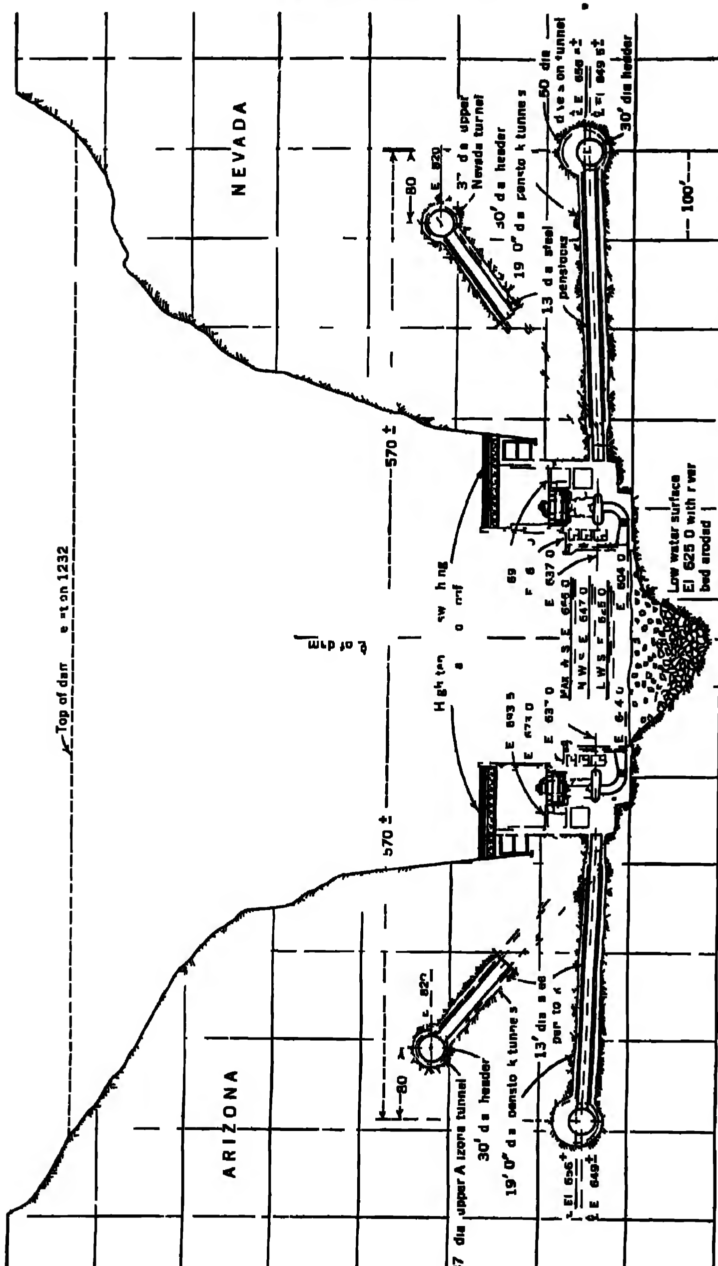

FIG 86 Hoover Dam development cross section through powerhouses in gorge just below dam

Dam on the Colorado River, Nevada-Arizona (Fig. 8), and Grand Coulee on the Columbia River, Washington (Fig. 9).

Although at the time a reservoir plant is installed market conditions may not make it practicable to install capacity on a peak-load basis, the possibility that such conditions may soon change should be considered. If the incremental cost of installation is low, provision should usually be made in the design for the later installation of a large amount of additional capacity.

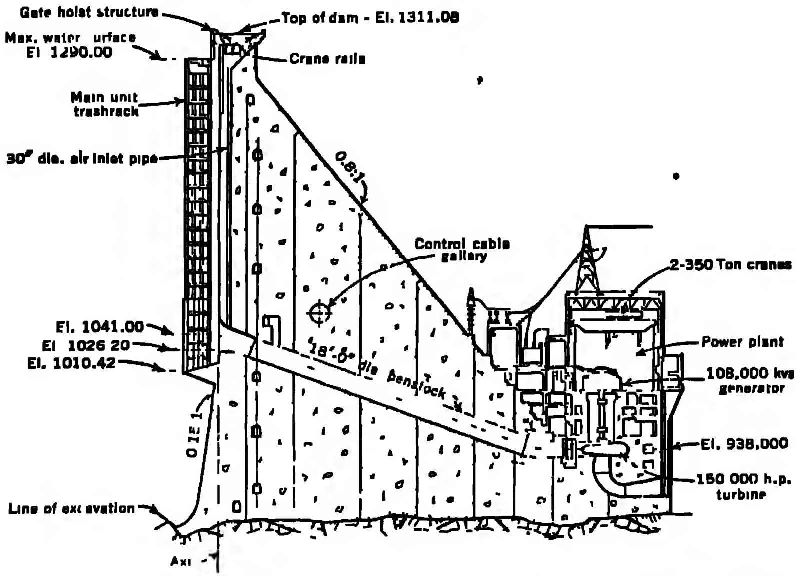

FIG. 9. Grand Coulee development, Columbia River, Washington. 823,000 kva (1944), 330 ft head, live storage 5,350,000 acre-ft (U. S Bureau of Reclamation.)

6. Peak-load Hydro Plants. A peak-load hydro plant is one designed and constructed primarily for taking care of the peak loads of a power system. Run-of-river hydro plants with pondage operate both as peak-load and base-load plants as river flows permit. A large pond or reservoir is essential, and extensive seasonal storage is usually provided. A characteristic of plants of this type is a high basis of installation in relation to stream flow. They frequently have large seasonal storage and relatively high heads and are likely to be located on small watersheds. They store up the water and clip the peaks off the top of the load curve. Plant discharge is often on a basis of 6 to 10 sec-ft per sq mi of watershed area. Figure 10 shows diagrammatically a series of peak-load plants of the Georgia Power Company.

Other illustrations of peak-load plants are Bagnell, Mo. (Fig. 11), Harriman, Vt., and Wallenpaupack, Pa.

7. Pumped Storage Plants for Peak Loads. Pumped storage hydro plants are peak-load plants that pump all or a portion of their own water

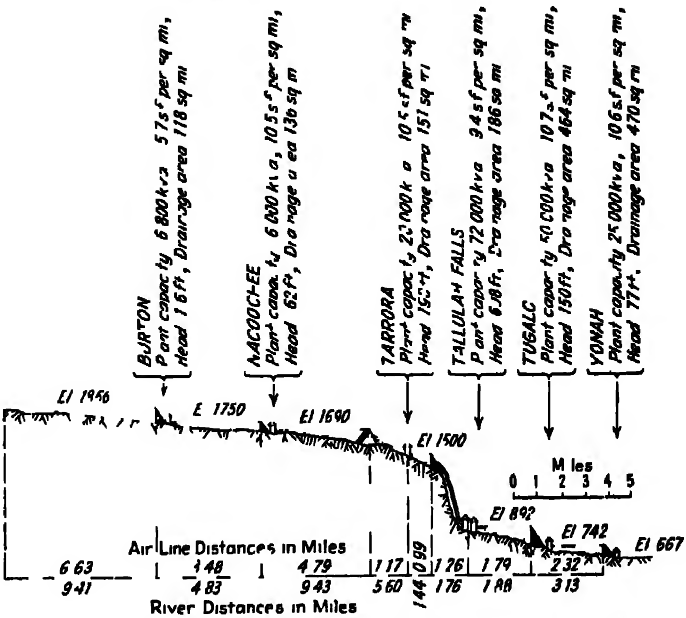

Fig. 10 Peak-load hydro plants of Georgia Power Company on Tallulah and Tugalo Rivers. The two power plants are on the Tugalo River and the others on its tributary the Tallulah.

Fig. 11 Bagnell development, Missouri. A peak-load plant, head 90 ft, installation 129,000 kw. Note omission of powerhouse superstructure. (Union Electric Light and Power Co., St. Louis, Mo.)

supply. Essentially they consist of a tailwater pond, which may be replaced by a river or a natural lake, and a headwater pond. During times of peak load, water is drawn from the headwater pond through the penstocks to operate hydroelectric generating units over the peak of the load curve. Dur-

FIG. 12. Rocky River pumped storage plant near New Milford, Conn., 24,000 kw, 230 ft head. (Fairchild.)

ing the off-peak hours, pumps are operated to shunt the water back from the tailwater pond to the headwater pond. Power for operating the pumps is furnished by off-peak steam-generated energy, sometimes supplemented by secondary hydro energy. For heads up to about 300 ft, it is feasible to use the same unit for both pumping water and generating power. This demonstrated fact will enlarge the field of application of such plants [1].

Reservoirs for such developments are merely large enough to provide for the plant's operation over the top of the peak load of the system it is de-

signed to serve, with some margin to permit using the plant for short-time breakdown service. A reservoir capacity that will permit full-capacity operation of the plant for 4 to 10 hr is usual. Occasionally seasonal storage

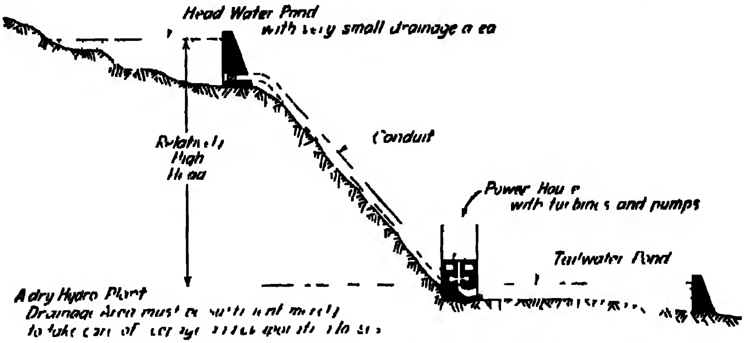

FIG. 13 Pumped storage hydroelectric plant installed for peak-load purposes only. Headwater and tailwater ponds sufficent for daily or weekly cycle of operations

has been provided for the benefit of other hydro plants on the river into which the discharge from the plant finally finds its way. The Hengstey plant in the Ruhr, Germany, and the Rocky River plant (Fig. 12) in Con-

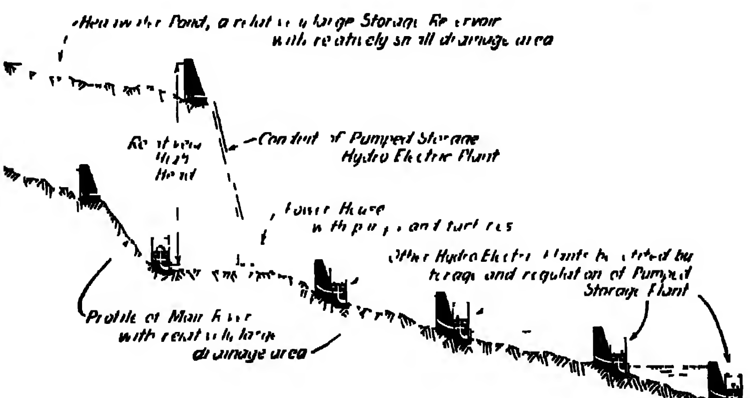

FIG 14. Pumped storage hydroelectric plant intended for regulation for peak-load purposes Headwater pond is a large storage reservoir, but drainage area may be very small Tailwater is a relatively large river and is the headwater of a hydro plant having a relatively large drainage area. The pumped storage provides peak-load service and also seasonal regulation for the other plants.

necticut are examples of pumped storage hydroelectric plants. Pumped storage plants are unique among hydroelectric plants in that practically no water supply is required. After the headwater or tailwater pond is once

filled, only enough inflow is required to take care of evaporation and seepage losses. Figures 13 and 14 show diagrammatically two types of pumped storage hydro plants.

8. Successful Pumped Storage Hydro Plants. The utilization of pumped storage hydro plants for carrying the peak loads of a system and for decreasing the operating costs of existing steam plants by giving them higher capacity factors is not new. In Europe, at least 40 such plants are in successful operation, the earliest of which were constructed before 1900 [2].

The Niederwartha plant near Dresden, Germany, has an installed capacity of 84,000 kw with a head of 460 ft and tailwater and headwater ponds each with a capacity of about 70,000,000 cu ft, or only enough to permit operation of the plant at full capacity for about 7½ hr. The plant serves to cut the peak off the load curve which would otherwise have to be carried by the Dresden steam plants, and off-peak energy from these steam plants is used for pumping.

The Hengstey plant in the Ruhr, Germany, adjacent to the coal fields, has an installation of 140,000 kw with a head of 520 ft. There is a small pond at the headwater (about 40,000,000 cu ft) and another at the tailwater. During the off-peak night hours, the water is pumped from the tailwater pond to the headwater pond, using off-peak steam-generated energy. During the on-peak day hours, the water is shunted back through the hydroelectric generating unit to the tailwater pond again.

Another type of pumped storage plant is constructed to have a seasonal regulating effect on other plants farther down on the watershed. The headwater of such a plant consists of a large storage reservoir. The Rocky River plant (Fig. 12), for instance, which has a capacity of 24,000 kw and a head of 230 ft, could operate 6 hr each day at plant capacity for 6 consecutive months without any pumping. The natural inflow to the reservoir may be insignificant, or it may furnish a material part of the water supply required. The tailwater of such a plant is either a relatively large river or the headwater of another hydroelectric plant on that river, and the discharge from the pumped storage plant may pass through a number of hydroelectric plants. The discharge is maximum at seasons of the year when the flow in the main river is minimum. Thus, the plant operates to increase the firm capacity of plants on the main river. The Rocky River plant, for instance, although its installed capacity is only 24,000 kw, creates a total firm capacity of 45,000 kw. In this respect it operates much the same as any storage reservoir of which the discharge is utilized for producing power. The essential difference is that a part of the stored water must be pumped from the main river during periods of high water.

The economic analysis of a project of this sort is identical with that for any power project with storage, except that the necessary pumping involves an additional operating expense.

Pumped storage peak-load hydro plants have one peculiar advantage over the usual peak-load hydro plant on a reservoir outlet. In the latter type, if the reservoir is drawn too low, firm capacity will be sacrificed, whereas, with

the pumped storage type of plant under such conditions, firm capacity can be maintained by additional off-peak pumping.

9. Special Application of Peak-load Hydro Plants to Systems with Sharp Peaks. Peak-load hydro plants are especially useful in systems subject to sharp and unexpected short-time peaks, as in some metropolitan loads. In such systems, a dark afternoon may cause coincidence of peak lighting loads, peak manufacturing loads, and peak transportation loads. The result is a sharp short-time peak (see Fig. 3, Chapter 14). A load curve of this sort necessitates a larger percentage of reserve capacity than is required by most power companies. If generation is 100% steam, a large part of the reserve must be kept hot and turning over, ready to pick up any unusual load. This in turn means that the company must keep reserve boiler capacity hot to furnish steam in the event of such a demand. Consequently, to serve and to be ready to serve such unusual peaks by means of steam electric power is very expensive. Peak-load hydro plants, on the other hand, are particularly adapted to perform such service and can frequently do it at much less cost.

10. Functions of Peak-load Hydro Plants. Referring to Fig. 6, Chapter 14, it will be noted that in the given week the 38,500-kw hydro plant operates as a peak-load plant clipping off the peaks of the load curve and thus permitting the steam plants to operate lower down on the load curve at a higher capacity factor and at lower unit production cost. However, this particular hydro plant is a run-of-river plant with large pondage, and it would operate exactly in the manner shown only at a time of coincidence of minimum December river flow and maximum demand week. Most of the time a great deal more water is available (the average annual capacity factor is around 50%), and the plant then operates farther down on the load curve. When full plant discharge is available, it operates on the base 24 hr a day and the older steam plants take the peaks.

In many systems purely peak-load plants prove economical, particularly in connection with storage projects. Thus, in the system represented in Fig. 6, Chapter 14, assume that, as the load increases in future years, storage is provided on the headwaters of the river system to such an extent that during the minimum December flow and maximum demand week represented by Fig. 6, Chapter 14, the plant has available, say, 1,200,000 kw-hr instead of the 438,000 kw-hr which is available without storage. The net effect of the increase in load and this additional energy is to leave at all times the sharp peaks of the load curve projecting above the band that can be served by this plant.

Consequently, these peaks might be served by installing a peak-load hydro plant at the reservoir. In many cases, the additional expense of such a peak-load hydro plant need not exceed $70 per kw of installed capacity (see Section 2, Chapter 13). This low additional amount results from the fact that the cost of dams and reservoir for storage is already incurred and the additional sum is for intake, conduits, powerhouse, and equipment. Chapter 13 deals more particularly with the costs of installation. Such installation should

seldom be made until practically all the installed capacity can be firm on the load curve.

Even if the entire expense of the reservoir must be borne by the peak-load hydro plant, the low capacity factor and consequent high basis of installation in relation to average annual stream flow would frequently assure a low capital cost per kilowatt.

The prime function of such plants is to carry short-time and unusual loads and to serve in case of need as an instantly available reserve capacity for the system; but, when operating as reserve, such plants may operate farther down on the load curve, drawing down the reservoir for this purpose. Such peak-load hydro plants permit the better steam plants to operate at more efficient capacity factors; they obviate the necessity for retaining in service so many antiquated high-production-cost steam plants and for carrying so many boilers hot and so many steam units in hot reserve.

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CHAPTER 12

COST OF STEAM POWER*

1. General. In the United States there is about two and one-half times as much steam-generating capacity as there is hydro capacity in central stations. Modern power-supply systems almost always include at least some steam plants and are frequently supplied mostly by steam power. Consequently, it behooves the hydroelectric engineer to have a general knowledge of steam plants and the cost of steam-generated electric power.

Internal-combustion engines as prime movers for the production of electric power have their field for small isolated installations. They are also useful for reinforcing large power supply systems near the ends of long distribution lines.

However, the relative importance of the oil engine in the central-station field is comparatively so small that no space will be devoted to the subject in the present work. More than 70% of all central-station capacity is steam, and the remainder consists almost entirely of hydroelectric plants.

2. Improvement in Steam Plants. The technical advance in the design of steam plants has been very rapid in recent years. In 1920 the public-utility plants of the United States required an average of 3 lb of coal to produce 1 kw-hr. In 1946 an average of only 1.33 lb of coal was required to produce 1 kw-hr., and some efficient plants and all new plants averaged considerably less than 1 lb per kw-hr. In addition, old steam plants which were thought to have very nearly completed their useful lives have been rejuvenated by the superposition of high-pressure units, thus becoming low-cost producers of energy.

3. Capital Cost of Steam Plants. The capital cost of steam plants is dependent on size of installation, cost of fuel, availability of condenser water supply, and design, and also on the price level for the location and the period of construction. Thus, a highly efficient plant located in a territory where fuel is relatively expensive might advisedly cost considerably more per kilowatt of installation than a less efficient plant located in an area where fuel is very cheap. Other things being equal, a large plant costs less per kilowatt of installation than a small one. Similarly, other things being equal, a plant located on a river where an adequate supply of condenser water is available will cost less than one located on a divide where a spray pond must be utilized for cooling the condenser water.

* The authors have used freely in this chapter tables and figures from Ref. 4, where applicable.

In general, the cost, in 1947, of a complete steam plant in the United States varied from $100 to $200 per kw of installed capacity. A large efficient steam plant (as, say, 3 units of 165,000 kw each) cost about $150 per kw of installed capacity and a small efficient steam plant (as, say, 3 units of 10,000 kw each) should cost about $170 per kw of installed capacity.

The initial capital cost per unit of capacity is frequently higher than the above because of the provision of land, water passages, fuel-storage and fuel-handling equipment, foundations, etc., for future units. Table 1 gives over-all cost data for nine actual steam plants.

TABLE 1

COMPARATIVE INVESTMENT AND PRODUCTION COSTS FOR NINE REPRESENTATIVE STEAM PLANTS

Plant	*A*	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*
Capacity, kw	150,000	60,000	335,000	187,000	235,000	80 000	30,000	180,000	145,000
Number of units	3	2	8	6	4	2	2	6	4
							$124 94	$125.74	$133.55
Fuel cost per million Btu	0.15	0.16	0.18	0.15	0.15	0.095	0 10	0.18	0.125
Fuel cost, mills per net kw-hr	2.32	3.44	3.60	3.57	3.11	1 97	1.32	3.01	2.61
Other costs, mills per net kw-hr	1.22	0.86	1.54	0.76	0.75	0.93	1.37	0.99	1.28
Fixed charges per kw-hr, 13½% fixed charges and mills, 45% capacity factor	3.35	3.60	3.09	4.15	3.91	4.18	4 26	4.30	4 56
Total cost, mills per kw-hr	6 89	7.90	8.83	8 48	7 77	7.08	6.95	8.89	8.45
Cost of coal per net ton on assumption of 13,000 Btu per lb	3.90	4.16	4 68	3.90	3.90	2.47	2.00	4.68	3.25

4. Fixed Charges on Steam Plants. Some difference of opinion exists as to the proper rate of fixed charges for both steam and hydroelectric plants. The elements of annual cost which, added together, form the fixed charges on a steam power plant are: cost of money, taxes, and insurance (usually lumped together for convenience); depreciation, and obsolescence.

Any given case should be carefully studied to determine the rate of fixed charges to use, but, on the basis of the discussions herein, fixed charges on steam plants will be taken as follows for illustrative purposes in the present text:

Cost of money	7.0%
Taxes and insurance	2.0
Depreciation and obsolescence	4.5
	——
Total annual fixed charges on steam plants	13.5% of capital cost

5. Cost of Money. "Cost of money" should be the average percentage return which investors in public-utility property have (considering all pertinent conditions) a right to expect from this investment. It is the "fair

return" which the courts have talked about. This is almost axiomatic, because, if the actually realized return falls permanently below a fair return rate, investors will not advance money for providing additional facilities. Thus, in the long run, if the public desires the facilities, it must necessarily permit a fair return.

Some government projects have used interest on government bonds as the cost of money but there a fallacy is involved because when the government elects to construct a power plant, it must forgo the income that it would otherwise obtain in the form of taxes.

In order to determine the true cost of money for a proposed government project some such setup as the following should be made. (It is herein assumed that government construction and operation are just as efficient as if the project was privately owned and that in each case the proposed project would be loaded to the same extent.)

COST OF MONEY TO THE GOVERNMENT FOR A POWER PROJECT AMOUNTING TO $1 000 000 (THE TERM "GOVERNMENT" AS USED HEREIN MEANS ALL GOVERNMENT—FEDERAL, STATE, AND LOCAL)

(1) Interest on government investment in power project 3% on $1 000 000	$30 000
(2) Taxes on privately owned public utility which government will have to forgo if it builds and operates the project itself, 15% of revenue (in many cases this is more than 16%) which on a 1 to 6 ratio would be 15% of $166 700	25 000
(3) Individual federal income taxes of investors on dividends and bond interest of alternative privately owned project which government would have to forgo if it builds and operates the project itself. Since such investors are ordinarily in the higher-than-average income groups this will probably average 15% of the 7% return received by them or 1.05% of $1 000 000	10 500
(4) State and local taxes on the income of investors which government will have to forgo if it builds and operates the project itself. In Pennsylvania this takes the form of a personal property tax, 0.4% state plus 0.4% county on values or, 0.8% on $1 000 000	8 000
Total annual cost of money to Government for $1 000 000 power project	$73 500

Hence on this basis the true cost of money for the government project is 7.35%, as compared to 7% for the privately owned project.

Item 3 is the writer's estimate as he does not know of any statistics on the subject. In the case of any individual investor, this item would be the highest increment of the income tax which he pays. There is also the question of multiple taxation in some cases, which would be very difficult to evaluate. Item 4 is precise for the Commonwealth of Pennsylvania, except that it does not apply to the securities of corporations incorporated in Pennsylvania, from which a more or less equivalent tax is collected. In other states the equivalent tax varies greatly in both directions.

> Although it is believed that the foregoing tabulation approximates the truth in many cases, it is presented merely to illustrate one of the setups which the government authorities should make as the result of extensive study to determine whether or not it is advisable to construct a government project.*

Necessarily "cost of money" is not a fixed rate. It varies according to time, location, and the financial standing of the organization. In preliminary setups for established organizations of good standing, 7% of the capital cost per year is usually considered a proper cost of money and has been generally used in illustrative examples in this book.

For well-established power companies in the United States the cost of money (represented by earnings on bonds, preferred stock, and common stock, plus expense of financing) in normal times will be from 6½ to 7½%, but temporary financial conditions may cause considerable variation. If the financial condition of the power company is not sound, or if present or prospective government competition is a factor, the cost of money may be so much greater than what the public-service commissions would consider a fair return on capital that additional capital expenditures are practically prohibited for the time being.

All that has been said above about the cost of money is applicable not only to steam plants but also to hydroelectric plants and, in fact, to all the capital expenditures of public utilities.

6. Taxes and Insurance. In determining the proper cost of taxes to use, it is not usual or desirable to include income taxes, as these are dependent on earnings; merely the local taxes on land and property, gross-receipt taxes, etc., are commonly included. Manifestly, taxes will vary materially with location, and as steam plants are generally located at or near a center of population, their taxes will usually be at a higher rate than those for hydro plants, which are likely to be located in remote rural sections.

Actual taxes paid are of interest in this connection. Current total annual taxes paid by several large systems as a percentage of fixed assets are as follows:

Commonwealth and Southern Power Corporation—1.8% of fixed assets (13.5% of revenue)
Niagara Hudson Power Corporation—2.8% of fixed assets (18% of revenue)
United Gas Improvement Company—2.3% of fixed assets (12.5% of revenue)
Standard Gas and Electric Company—2.3% of fixed assets (13.5% of revenue)

All taxes, both property and income taxes, are included in the above. It might be pointed out also that the property tax rate is usually much higher in cities than in rural communities.

Insurance rates vary less widely but are confined to the insurable items in the plant. In illustrative examples herein, the annual cost of taxes and insurance is assumed as 2% of the capital cost of a steam plant.

* *Trans. A.S.C.E.*, Vol. 104, p. 1151, 1939 (Justin on Cost of Energy Generation).

7. Depreciation and Obsolescence. When a man builds any kind of plant or machine, he knows that its useful life is limited. Accordingly, it is necessary for him, at least theoretically, to make annual payments to a reserve fund large enough to cover renewals and replacements and also to finance a new plant of the same size when the present one is no longer useful. Depreciation and obsolescence are in reality separate factors; depreciation is due to physical deterioration, whereas obsolescence is dependent on the advance in science or in some other industry. Thus, an interurban trolley line may have all its track and equipment in excellent condition but may be obsolete and have to discontinue operations because of the development of interurban bus lines.

Theoretically it should be a simple matter to determine precisely the proper allowance for annual depreciation. All one needs to do is to make sufficient payments to a reserve fund so that necessary withdrawals from that fund will keep the given equipment in perfect operating condition for all time. Practically, the determination of a proper allowance for annual depreciation is not so simple. The life of any given piece of equipment cannot be accurately determined in advance, and even the operation of compound interest, usually assumed for the reserve, is not at all certain from a practical standpoint.

Obsolescence is still more uncertain from both a theoretical and a practical standpoint, because no one can foretell just when an advance in the science or possibly an advance in some other industry may make any particular equipment entirely obsolete. Consequently, no fine degree of precision is warranted in determining the proper annual payments to reserve to take care of such items. The best guide is experience.

In this connection, the actual practice of soundly operated systems is of some interest. The following are approximate current annual payments to "reserve for renewals and replacements" or "depreciation reserve" for the purpose of covering depreciation and obsolescence in several of the country's major systems:

Commonwealth and Southern Power Corporation—1.5% of fixed assets (11% of revenue)
Niagara Hudson Power Corporation—2.0% of fixed assets (12.8% of revenue)
United Gas Improvement Company—1.7% of fixed assets (9.1% of revenue)
Standard Gas and Electric Company—2.0% of fixed assets (12.5% of revenue)

The above are over-all rates on all the property of the public utility. There is, of course, a wide variation in the depreciation and obsolescence rates on the various items. Thus, the rate on a modern office building would be much less than a steam power plant, and the rate on a hydroelectric plant (with its usually large investment in lands, riparian rights, and dams) probably much less than for either office buildings or steam plants.

Although essentially different, these two factors, depreciation and obsolescence, are bracketed together here because both are dependent on the time element and also because it is usual practice with public utilities to utilize a

single reserve for both factors. The estimated useful life expectancy of the various elements of a steam plant, as used by many engineers, is given in Table 2.

TABLE 2

Useful Life Expectancy of the Principal Parts of a Steam Power Plant

Description	Probable Life, years
Accumulators	15
Boilers—water tube	20
Boiler accessories	20
Breechings—steel	10 30
Buildings	
Brick	30
Wood or wood frame	20
Cables and feeders	15 25
Coal and ash machinery	20
Compressors—air	20
Condensers	20
Cranes	30
Economizers and air preheaters	15
Electric generators	20
Electric motors	20
Engines—small steam	15
Feedwater heaters	20
Fences	12
Foundations	Same as life of equipment supported
Fuel-oil-handling equipment	20
Furniture and fixtures	15
Pipe and pipe covering	15 25
Pumps—reciprocating	15 20
Pumps—centrifugal	20
Stacks—brick or concrete	30
Steam turbines	20
Steel	12 15
Stokers and other fuel-burning equipment	20
Superheaters	20
Switchboards and switchboard equipment	20
Tools and shop machinery	15
Transformers	15

Both the sinking fund and fixed-rate methods are used for determining the proper annual payments to the reserve for depreciation and obsolescence or "reserve for renewals and replacements." Engineers usually favor the sinking-fund method, which is utilized herein. Figure 1 gives curves for determining the annual payments to depreciation reserve that will make reserve equal capital cost in a given number of years. Table 1 of Chapter 16 gives the same data but for a wider range of money rates.

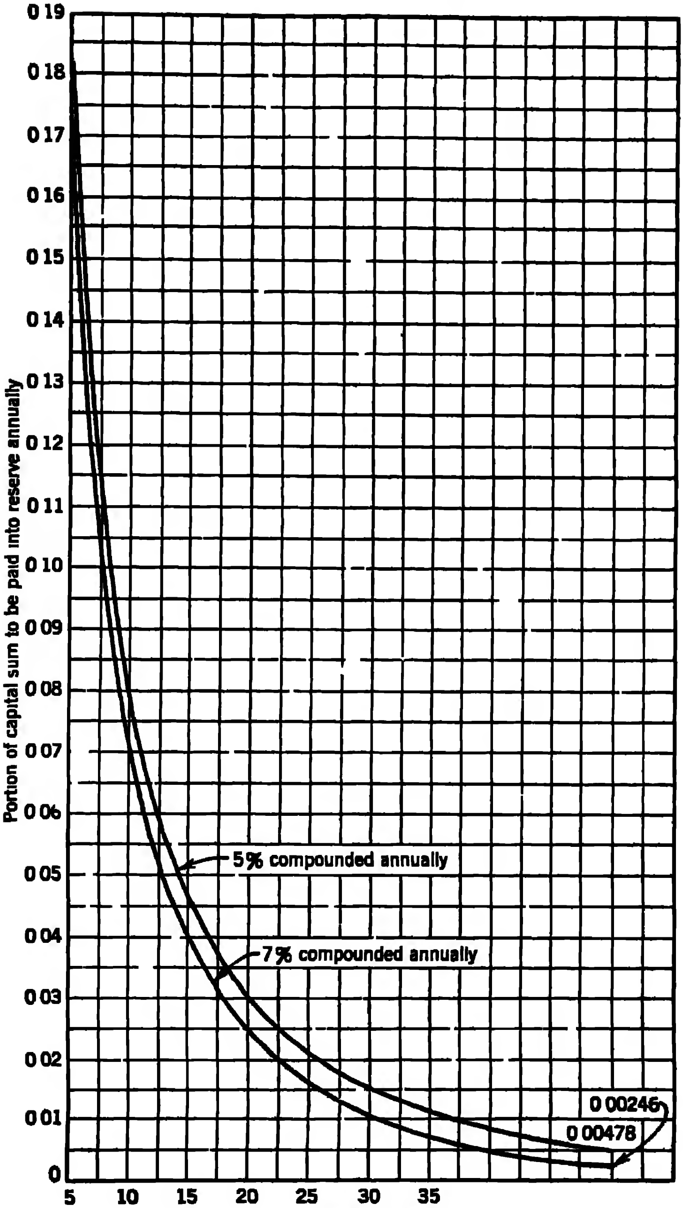

FIG 1 Curves for determining annual payments to depreciation reserve that will make reserve equal capital cost in given number of years

Both depreciation and obsolescence considered, it seems to be the consensus of the best informed that the over-all useful life expectancy of steam plants is 15 to 16 years. On this basis and with the reserve fund money operating at 5% compound interest, annual payments to the reserve have herein been taken at 4.5%.

8. Capacity Factor. The capacity factor at which a power plant operates has a major influence on the cost of power. Some engineers speak of the load factor of a power plant when they mean the capacity factor. Load factor concerns the load of the system or plant, whereas capacity factor concerns the capacity of the plant. Inasmuch as power plants are seldom loaded to the limit of their capacity, there is usually a material difference between the values of these two factors. The capacity factor for any given period of time (day, week, month, or year) may be defined as the ratio of the energy that the plant actually produced to the energy that it might have produced if operated at full capacity throughout the period.

Thus, if during a given week the peak load on a power plant with a capacity of 100,000 kw was 65,000 kw, and if the energy produced by the plant was 6,720,000 kw-hr, the *capacity factor* for that week (168 hr) would be

$$\frac{6{,}720{,}000}{100{,}000 \times 168} = 0.40 \text{ or } 40\%$$

During the same period, however, the *load factor* on the plant would be

$$\frac{6{,}720{,}000}{65{,}000 \times 168} = 0.616 \text{ or } 61.6\%$$

The term "plant factor" as used by many engineers is identical with "capacity factor" as here defined.

9. Actual Capacity Factor of Power Plants. One advantage which steam plants inherently possess is that they may be operated at almost any necessary capacity factor up to the practicable limit, whereas the annual capacity factor at which hydro plants operate is usually limited by the variation in water supply (except in the case of hydro plants like those at Niagara, where installation is less than minimum stream flow and where annual capacity factors may exceed 95%). Theoretically, a steam plant might operate at a 100% annual capacity factor, but practically, because of the necessity for an annual take-down period, the maximum annual capacity factor is much lower and the usual practicable maximum annual capacity factor does not exceed 80%.

Actually the average annual capacity factors of steam plants in the United States is somewhat lower than those of hydro plants, as shown by Tables 3 and 4.

10. Decline in Annual Capacity Factor of Steam Plants. When a new steam plant is constructed, it is usually one of the most efficient steam plants in the system, and, accordingly, as much of the system load as practicable is thrown on it during the early years of its life or until additional

TABLE 3

ANNUAL OVER-ALL PLANT FACTORS OF ALL PLANTS OPERATED BY ELECTRIC UTILITIES AND OTHER ORGANIZATIONS PRODUCING ELECTRICAL ENERGY FOR PUBLIC USE *

Year	All Hydro Plants	All Steam Plants
1926	47.4	30.8
1927	49.2	30.5
1928	51.6	29.6
1929	48.0	32.4
1930	43.4	30.0
1931	37.5	27.7
1932	40.8	21.4
1933	41.1	22.0
1934	40.0	25.1
1935	46.7	26.3
1936	45.8	32.2
1937	49.7	34.2
1938	48.5	30.6
1939	45.9	35.6
1940	48.5	38.7
1941	50.4	44.7
1942	59.1	45.3
1943	62.9	50.6
1944	59.1	52.0
1945	61.9	47.3
1946	60.1	47.5

* From data in Federal Power Commission publications S-48 and S-50.

TABLE 4

APPROXIMATE OVER-ALL PLANT FACTORS * OF THE ELECTRIC LIGHT AND POWER INDUSTRY IN THE UNITED STATES

Description	1926	1927	1928	1929	1930	1931	1932	1933	1934	1935	1936	1937
All hydro plants	45	46	49	48	41	35	39	40	39	45	45	48
All steam plants	30	29	29	31	29	28	21	22	25	26	32	34

* Percentage computed from basic data in *Electrical World*, Jan. 15, 1938. Plant factor is the same as capacity factor.

efficient or more efficient plants are constructed. From that point on, the annual capacity factor of the plant generally declines. Figure 2 shows the influence of age on the annual capacity factor of steam plants.

This decline of annual capacity factor with age, which occurs with steam plants (and not with hydro plants), is one evidence of obsolescence and is an important factor in determining the total cost of steam power, as the relatively high annual cost of depreciation and obsolescence indicates.

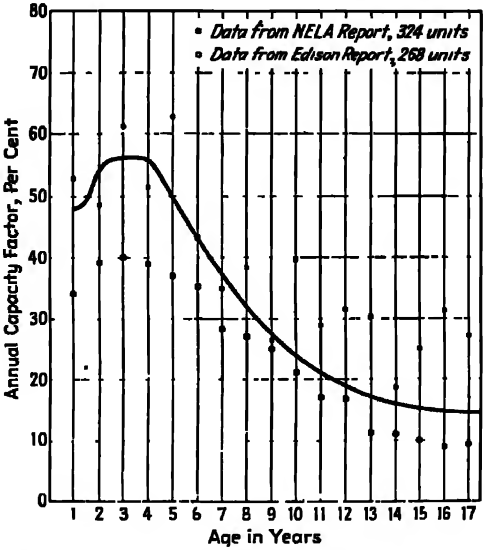

FIG. 2. Effect of age on annual capacity factor of steam plants.

11. Operating Costs of Steam Plants. Operating costs of steam plants are sometimes stated as averaging so many mills per kilowatt-hour. This is an extremely inexact method and is practically useless for economic studies unless accompanied by a statement of the operating statistics of the plant to which it applies.

The facts of the matter are that a substantial portion of the annual operating and maintenance costs is incurred just to keep the plant ready to run if required and that the remainder of the operating and maintenance cost is very nearly proportional to the number of kilowatt-hours produced. Thus, a steam plant held in reserve and utilized to carry unusual peak loads may have an annual capacity factor of, say, 10% and show an annual operating cost of 10 mills or more per kw-hr, whereas the same plant operated at an annual capacity factor of 50% may show an average operating cost of 3 mills per kw-hr.

Figure 3 shows the effect of annual capacity factor on the total cost of steam-generated energy.

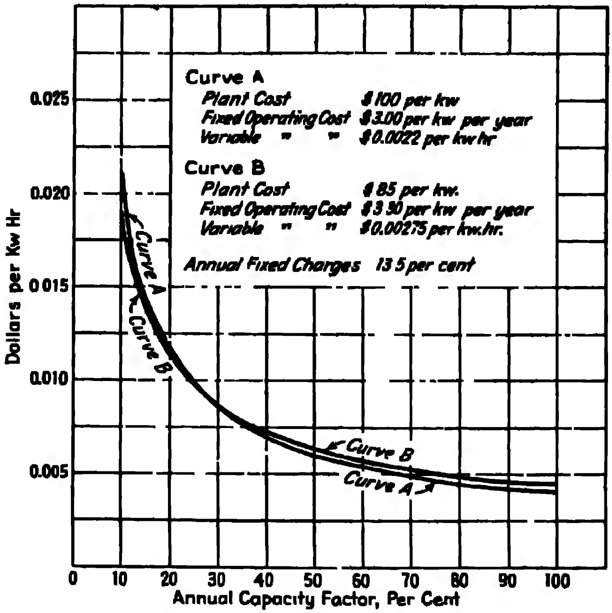

FIG. 3 The effect of annual capacity factor on total plant power costs.

12. Fixed and Variable Cost of Operating. To facilitate economic comparisons between the various steam plants of a system or between steam plants and hydro plants, the total annual operating cost of a steam plant may be allocated in accordance with the following formula: *

$$A = K_1 + K_2 + (K_3 \times \text{kw-hr})$$

where A = the total operating cost in dollars for the year;

K_1 = that portion of the total operating cost which is determined by capacity;

K_2 = that portion of the total operating cost which is determined by the peak prepared for;

$K_3 \times$ kw-hr = that portion of the total operating cost which is proportional to the number of kilowatt-hours produced.

For preliminary economic studies it is usually sufficiently exact to combine K_2 with K_1. This would be precise only if the plant were fully utilized each year. The formula then becomes

$$A = K_1 + (K_3 \times \text{kw-hr})$$

It is frequently utilized on a per kilowatt of installed capacity basis, and K_1 is then spoken of as "the fixed cost of operating per kilowatt of capacity,"

* This method of cost allocation closely follows one devised by N. E. Funk and is described in more detail in Chapter 6 of Ref. 4.

and $K_3 \times$ kw-hr is referred to as "the variable cost of operating" because it is dependent on the number of kilowatt-hours produced per kilowatt of capacity.

The approximate accuracy of the above formula can be demonstrated by taking the production and operating cost records for a given steam plant for a number of years and plotting net kilowatt-hours produced as abscissas against total operating cost (equating to the same unit cost for fuel and labor). It will be found that the points are very nearly on a straight line that intersects

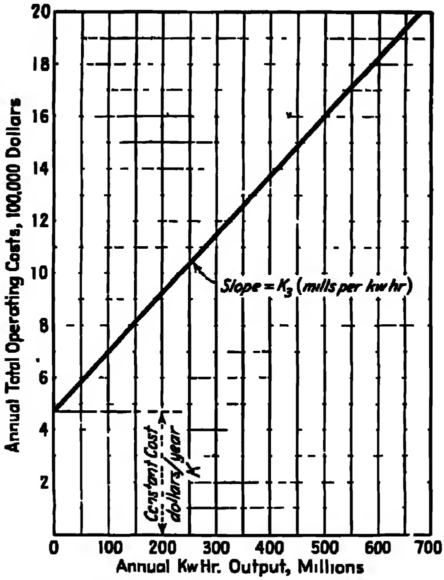

FIG. 4. Method of dividing operating costs, etc.

the Y axis above the origin. The point on the Y axis shows the annual fixed cost of operating, and the difference between it and points on the slanting straight line indicates the variable, or incremental, cost of operating (as it is sometimes called) for any given annual output of energy. Figure 4 shows the results of such a plotting for a plant with an installed capacity of approximately 100,000 kw.

13. Actual Annual Costs at Steam Plants. From the preceding discussion it is evident that operating costs at steam plants will vary over a rather wide range. Table 5 gives an analysis of operating cost at ten typical steam power stations.

Plant A is an efficient high-pressure plant, and, although it was completed in 1929, very few plants show greater economy today.

It will be noted that the incremental costs vary all the way from a 1.58 mills per kw-hr (Plant I) minimum to a 4.12 mills per kw-hr (Plant G) maximum, and that the annual fixed cost of operating varies from a \$2.30

TABLE 5

ANALYSIS OF PRODUCTION COSTS OF TEN TYPICAL POWER STATIONS

Plant	A	B	C	D	E	F	G	H	I	J
Capacity, kw	60,000	120 000	100 000	188,000	70 000	180,000	70 000	28,000	21 000	80,000
Number of units	1	2	2	5	2	6	3	2	1	1
$K_1 + K_2$ cost per kw capacity	$2 30	$4 45	$4 00	$4 51	$3 96	$4 16	$11 44	$3 90	$3 20	$3 00
K_3 cost, mills per kw-hr	1 75	2 45	2 50	3 34	2 75	2 91	4.12	2 28	1 58	1 60
Coal cost per net ton	$4 00	$4 30	$2 25	$4.55	$4 00	$4 35	$2 25	$2 00	$1.90	$4 30
Btu per lb in coal (avg.)	14 000	13 800	10 500	13 800	14 000	13 800	10 800	10 800	10,800	12,900
Year built	1929	1926	1927	1924 1927 *	1925	1922	1918 1920 *	1923	1926	1937

* Additions to original station.

per kw of capacity (Plant A) minimum to a $11.44 per kw of capacity (Plant G) maximum.

To obtain the total annual cost, fixed charges must be added to the above, which, using a rate of 13.5% (see Section 4) and a capital cost of $100 per kw, would alone amount to $13 50 per kw of capacity per year.

Each particular situation requires investigation by engineers experienced in steam plant engineering in order to determine the actual or anticipated cost of steam-generated electric power. In general, assuming that units need not be smaller than 30,000 kw each, it is seldom necessary that the total cost of steam-generated electric power from new plants where the cost of 14,000 Btu per lb coal is $4 00 net ton should exceed $16.50 per kw per year plus 2 3 mills per net kw-hr generated per kilowatt of installed capacity. This figure includes fixed charges at the rate of $13 50 per kw per year.

14 Effect of Fuel Cost on Cost of Steam Power The effect of the cost fuel on the total cost of steam power is considerable but not as great as one unfamiliar with steam plant costs might expect Thus, the total cost per kilowatt-hour of steam-generated electrical energy (including fixed charges) at 50% capacity factor might be 6 mills for a modern steam plant with 14,000 Btu coal costing $4 00 per net ton at the plant. If the cost of coal is lowered from $4 00 per net ton to $3 00 per net ton, the total cost of energy at 50% capacity factor will be 5½ mills per kw-hr, a reduction of ½ mill per kw-hr The reduction in fuel cost has been 25%, but the reduction in total cost of energy has been only about 8⅓%.

With regard to the incremental cost of energy (the K_3 factor in the formula given in Section 12 of this chapter), the cost or K_3 factor is very nearly, but not quite, proportional to the cost of coal. This is because approximately 90% of K_3 is fuel cost, whereas only about 15% of the K_1 factor is cost of fuel.

It may be rather loosely stated that for a modern steam plant a change of $1 per net ton in the price of coal will result in a change of ½ mill in the total cost of electrical energy generated over any usual operating range. For the older, less efficient plants the difference will be greater.

The effect of variation in the cost of fuel on the total cost of electrical energy produced by a steam plant is shown in Fig. 5. The plant is a more or less typical modern steam plant having a capital cost of $100 per kw of capacity with fixed charges of 13.5% and using 14,000 Btu coal.

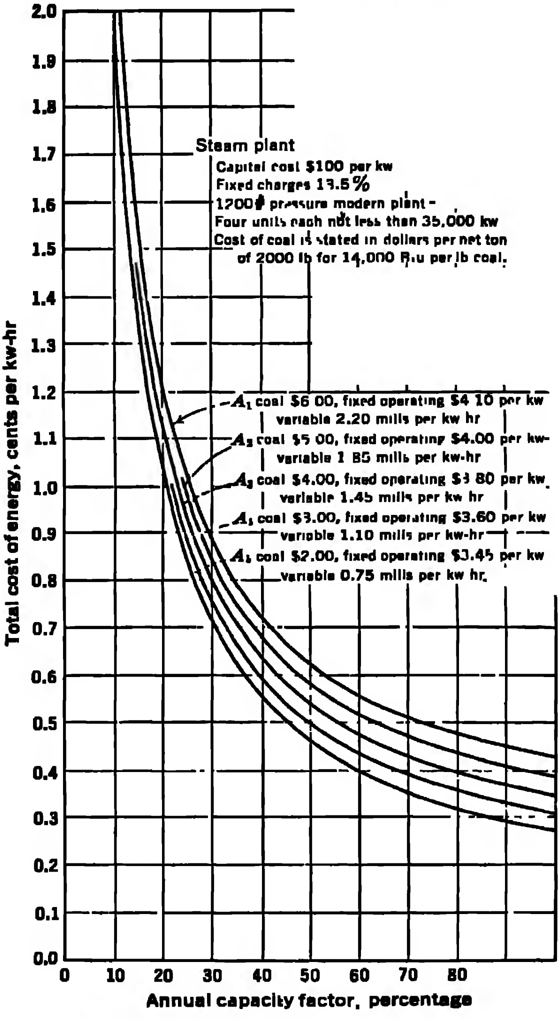

FIG. 5. Effect of cost of fuel on total cost of steam-generated electrical energy.

15. Fuel Oil and Gas Equivalent of Coal. If one is dealing with problems involving the use of fuel oil, it is often convenient to substitute the equivalent in tons of coal, as would be the case in using Fig. 5. One net ton of 14,000 Btu coal contains 28,000,000 Btu. Over-all efficiency when burning

fuel oil is not usually quite so great as when burning coal Hence, 1 bbl (42 gal) of fuel oil containing 6,000,000 Btu can be assumed to have an effective Btu content of 5,800,000, and 1 net ton of the coal considered in Fig. 5 can be taken as the equivalent of 28,000,000 ÷ 5,800,000 = 4 83 bbl of fuel oil containing 6,000,000 Btu per bbl (or 18,500 Btu per lb of oil weighing 7 7 lb per gal). For instance, if such fuel oil costs \$1 00 per bbl, it is equivalent to the coal of Fig 5 costing \$4 83 per net ton

If the fuel is gas, simply determine the cost of 28,000,000 Btu of gas and use Fig 5 in the same manner.

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CHAPTER 13

COST OF HYDRO POWER

1. Capital Cost of Hydroelectric Plants. The capital cost of hydroelectric plants varies over a much wider range than that of steam plants. In 1947 modern steam plants generally cost from \$100 to \$200 per kw of installation. On the other hand, the cost of hydroelectric developments runs all the way from \$150 to \$400 per kw of installation.

The capital cost of a hydroelectric development or project * includes the cost of all lands and riparian rights; the cost of railroad, highway, and other public-utility changes; and the cost of construction, engineering supervision, and overhead for all items of the development up to the low-tension side of the step-up transformers. The step-up transformers, high-tension switching, and transmission lines, etc., are not usually included in capital cost of the hydroelectric development. It is necessary to emphasize this definition because it is found that, though the above is generally accepted, some engineers include cost of transmission, and others, in speaking of the cost of a hydro development, may leave out such items as riparian rights and lands. Others may indicate an astonishingly low unit cost for a hydro development by allocating a large part of the cost of the dam and reservoir to some other function like flood control or irrigation. In the present work, where cost of the development or project is used, the total cost, as defined above, is meant. The allocation of such costs in accordance with benefits derived is a different problem.

The reason for the greater range of cost for hydro plants than for steam plants is not difficult to find. Only about 15% of the total cost of steam plants is influenced by topographical and geological conditions, whereas, with hydroelectric developments, 75 to 85% of the total cost is thus affected.

Factors affecting the capital cost of hydroelectric developments are:

(*a*) Riparian rights and control of site.
(*b*) Topography of the site.
(*c*) Geological conditions at dam site and reservoir site.
(*d*) Extent and value of lands flooded.
(*e*) Public utilities required to be relocated such as roads, bridges, railroads, pipe lines, and power lines.

* Wherever in this book the term hydroelectric project or hydro project is used it should be understood to mean a proposed hydroelectric development. Similarly, a hydroelectric development or a hydro development is a project which has actually been constructed.

(f) Towns and communities flooded.
(g) Quantity and character of stream flow, i.e., as steady flow, or subject to wide variation with extreme floods.
(h) Available head.
(i) Capacity of the development.
(j) Price level (prevailing costs of labor, materials and equipment at the time and place of construction).
(k) Skill of the engineer.
(l) Foresight of executives.
(m) Efficiency of the construction organization.

The above factors are not listed in the order of importance. It would be entirely impracticable to rank them, as the order of importance would change materially from project to project. Because there is usually not just a single site to be considered, but several, some of them on different watersheds, and also because the size of the development and head are usually open questions before construction, it is probable that, in most cases (h) the skill of the engineer, (i) the foresight of executives, and (j) the efficiency of the construction organization are the most important factors determining the capital cost of the development.

TABLE 1

FEATURES AND TOTAL CAPITAL COST OF REPRESENTATIVE HYDROELECTRIC DEVELOPMENTS

No	Water-shed Area, sq mi	Mean Annual Runoff, sec-ft per sq mi	Storage or Pondage in Per Cent of Mean Annual Runoff	Gross Head	Length of Conduits, ft	Present Installation, kw	Present Number of Units	Total Capital Cost per Kilowatt of Present Installation	Eventual Installation, kw
1	27,089	1.44	0.25	89	(D)	252,000	7—36,000	$197	396,000
2	26,108	1.44	0.25	55	(D)	168,000	6—28,000	171	336,000
3	[illegible] 400	1.[illegible]4	0.22	183	1,180 (P)	140,000	4—32,500	160	195,000
4	13,850	0.76	0.16	90	(D)	129,000	6—21,500	256	172,000
5	1,870	2.44	4.31	217	2,347 (P)	121,500	3		121,500
6	2,400	1.32	9.8	116	(D)	100,800	2—50,400	305	100,800
7	10,200	1.63	0.41	87	(D)	100,000	4—25,000	124	150,000
8	2,880	1.52	43.0	125	(D)	99,000	3—33,000	184	132,000
9	4,130	1.40	9.0	179	360 (P)	81,200	4		81,200
10	1,630	2.38	0.27	189	700 (P)	72,000	4		72,000
11	1,300	1.40	26.8	100		56,000	2—28,000	146	56,000
12	2,660	1.61	25.2	135		48,000	2—24,000	195	72,000
13	4,200	1.9	3.0	114	330 (P)	45,000	3—15,000	175	60,000
14	464			150		45,000	4—11,250		
15	175	3.10	34.1	600	25,836 (P)	45,000	2	178	45,000
16	3,930	1.40	5.95	61	(D)	33,000	3		33,000
17	4,140	1.40	9.1	52.8	(D)	20,300	3		20,300

(D) = Power plant at dam.
(P) = Penstock including flow line conduit.

TABLE 2

COST OF CONSTITUENT PARTS OF HYDROELECTRIC DEVELOPMENTS

Name	River	State	Date Completed	Annual Capacity Factor	Capacity in Thousands of Kilowatts at Switchboard	No of Units	Head, ft	Cost in Dollars per Kilowatt of Capacity					
								Dam	Waterway	Powerhouse and Equipment	Highways Railways, etc	Land and Water Rights	Total
Conowingo	Susquehanna	Md	1928	0.59	252	7	89	41	7	89	33	27	197
Safe Harbor	Susquehanna	Pa	1932	0.56	168	6	55	46		110		5	161
Saluda	Saluda	S C	1931	0.23	130	4	183	89		23	1	47	160
Jordan	Coosa	Ala	1928	0.46	100	4	87	92				32	124
Martin	Tallapoosa	Ala	1926	0.32	99	3	125	149				35	184
			1925	0.55	81	3	315	26	77	26	10	4	143
Ohio Falls	Ohio	Ky	1928	0.45	80.3	8	37	2		93			95
			1922	0.46	70	2	454	4	42	41	11	23	121
			1921	0.79	66.7	3	1149	18	152	42	2	2	216
			1908	0.94	65.0	6	465	14	64	34	2	0	114
			1931	0.55	60	2	1218	11	91	24	3	1	130
Rock Island	Columbia	Wash	1933	0.72	60	4	32	140		76		27	243
Gorge	Skagit	Wash	1924	0.49	55.5	3	270	80		64	43	1	188
			1928	0.53	50	2	2561	28	63	26	9	4	130
Chelan	Chelan	Wash	1927	0.66	48	2	410	37	60	46		50	193
Ariel	Lewis	Wash	1932	0.56	46	2	170	110	5	45	9	20	189
Bartletts Ferry	Chattahoochee	Ga	1926	0.55	45	3	114	94	7	43	4	27	175

Harriman	Deerfield	Vt	1924	0 36	43	3	360	86	73	38	28	14	239
Wallenpaupack	Wallenpaupack Cr	Pa	1926	0 22	40	2	370	32	55	64	3	76	230
Kerckhoff	San Joaquin	Calif	1920	0 56	39	3	350	21	84	37	2		144
Piney	Clarion	Pa	1925	0 22	28 8	3	80	204		152		14	370
Exchequer	Merced	Calif	1926	0 32	25	2	210	156	9	37	0	256	458
Twin City	Mississippi	Minn	1924	0 37	13 4	4	30	1	17	92			110
Glines Canyon	Elwha	Wash	1927	0 81	12	2	160	57	30	44		17	148
Mystic Lake	West Rosebud	Mont	1924	0 65	10 5	2	1050	18	93	50		11	172
Borel	Kern	Calif	1925		10		270	5	158	36	0	11	210
Lewiston	Clearwater	Ida	1927	0 21	10	2	36	134	55	79		68	336
Kern Canyon	Kern	Calif	1921	0 59	8 5	1	260	17	149	63	0	5	234
Sherman	Deerfield	Mass	1926	0 41	7	1	78	120	16	84		86	306
Green Island	Hudson	N Y	1923	0 37	6 6	8	14	4	63	201			268
Molly's Falls	Winooski	Vt	1926	0 11	5 0	1	350	40	43	38		51	172
			1937	0 59	3 0	1	41	83	18	74	34	32	241
Allegan	Kalamazoo	Mich	1937	0 43	2 4	3	16	62		108	10	74	254
Battle Creek	Battle Creek	Utah	1927	0 10	2 4	1	1790	1	56	43		1	101
Dam No 7	Kentucky	Ky	1928	0 44	2 0	3	18	2	24	205			231
Logan	Logan	Utah	1927	0 68	2 0	2	220	23	119	68		1	211
Alpine	Alpine Creek	Utah	1927	0 25	1 8	2	1920	1	74	57		1	113
Mottville	St Joseph	Mich	1923	0 24	1 7	4	13	175	2	98		52	327
Stairs	Big Cottonwood Cr	Utah	1927	0 43	1 5	2	370	90	69	52		1	212
Upper Amer Fork	Amer Fork Cr	Utah	1927	0 48	1 2	2		6	94	57			159
Big Bend	Rapid Creek	S Dak	1930	0 19	1 2	3	268	8	196	53		7	284
Lower Amer Fork	Amer Fork Cr	Utah	1927	0 47	1 0	2		11	104	60		2	177

From the above it is clear that hydro developments are quite individualistic and that problem and costs are sure to vary over a wide range.

Although it would be possible to give an extensive compilation of the cost of existing hydroelectric developments, it is believed that the list in Table 1 is fairly representative of American plants. Since the data are largely from the authors' files, designating numbers have been used instead of the actual names of developments.

In Table 2 * is given the cost of a number of hydroelectric plants with the total cost broken down (or *allocated*) among the various features of the development as dam, waterway (or conduit), powerhouse and equipment, highway and railroad relocation, etc., and land and water rights.

From the data in Tables 1 and 2 it is evident that there is nothing consistent about the cost of hydroelectric developments. It is almost literally true that each is a law unto itself. Furthermore there is no adequate simple yardstick for measuring the reasonableness of the cost of a hydro development. A development which cost $400 per kw would sound like a very expensive development. Although usually that would be true the development might include a large reservoir which "firmed" up the capacity of a number of plants down the river and then that cost might be entirely reasonable.

The total capital cost of another development might be only $120 per kw, which, offhand, sounds reasonable. If, however, it is a low-head, run-of-river plant without pondage, where the plant becomes frequently inoperative because of high tailwater, then the cost might, in reality, be unreasonably high and economically unjustified. The subject of the economic advisability of hydro installation is fully discussed in Chapter 15.

2. Increment Cost of Hydro Installation. As indicated in Tables 1 and 2, the cost of hydroelectric developments varies widely, and total cost may not bear any direct relation to the amount of installation. However, for any given site, certain portions of the total cost are at least roughly proportional to the capacity installed. In general these parts are intakes, conduits, powerhouse, and equipment.

Because the cost of these four items varies so nearly with installation capacity they may be termed the increment cost of installation. Thus, if the total cost of these items for some given plant is $600,000, and the installation is 10,000 kw, the increment unit cost of installation would be $60 per kw for that project. It is usually a fair assumption that additional units might have been added at this unit cost at the time the plant was constructed. Usually, when one speaks of the unit cost of powerhouse and equipment for low-head plants, he means the same thing as the increment unit cost.

One would expect to find that the increment unit cost of installation for high-head plants would be much less than for low-head plants. A careful study, however, has justified the conclusion that it is impracticable to find

* Table 2 is abstracted from a more extensive table on p. 1099 of Ref. 3, Section 15.

any general relationship between head and increment unit cost of installation [2].* In fact many of the very low-head plants were found to have among them the lowest increment unit costs of installation.

In general, in 1947, it was usually not practicable to design and construct a hydroelectric project having an increment unit cost of installation less than \$50 per kw. It may also be stated that it is seldom necessary for the increment unit cost to exceed \$100 per kw. When the initial construction includes tailrace and the whole or a portion of the substructure, the increment unit additional cost of adding additional capacity to the existing plant may be much less than the figures mentioned above.

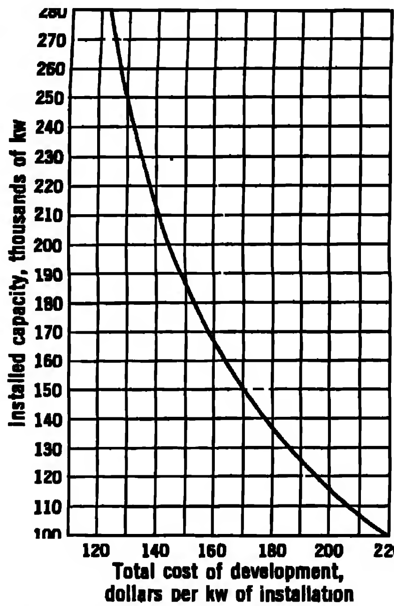

FIG. 1. Effect of increasing installation at a certain hydroelectric development on total unit capital cost.

3. Effect of Increment Unit Cost of Installation on Total Cost of Project. If a hydroelectric project is being considered for construction in any river, the site and practicable head will be determined as the result of field investigations and office studies. The next problem is to determine how much capacity to install. The cost of lands and riparian rights, of relocation of public utilities, and of the dam will remain practically constant regardless of the amount of capacity installed. The remaining costs or "increment cost of installation" (powerhouse and equipment including intakes and conduits) will vary almost directly with the capacity which it is decided to install. Consequently the total cost of the project may be expressed by the formula

$$C = F + IY \qquad [1]$$

in which C = total capital cost of the project;
F = that portion of the cost which is fixed for the given site and head regardless of installation;
I = increment unit cost of installation per kilowatt of capacity;
Y = number of kilowatts of capacity which it is proposed to install.

Figure 1 shows the effect of increasing amounts of installation on the total unit cost at a well-known development.

* See p. 151 of Ref 2 in the bibliography at the end of this chapter.

4. Cost of Bridgeport Hydro Plant. At the site of the Bridgeport hydroelectric project, which has ample pondage but no storage, the determined average net head is 50 ft and the costs which are fixed (dams, lands, riparian rights, relocation of public utilities, etc.) amount to \$6,000,000. This is factor F in Eq. 1 above. It has also been found for this site that the value of I (increment unit cost of installation per kilowatt of capacity) is approxi-

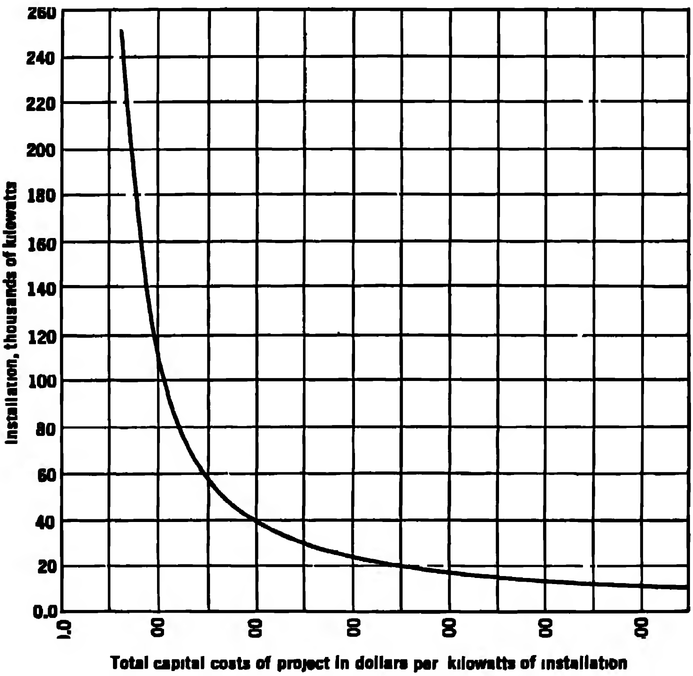

FIG. 2. Bridgeport hydro project. Effect of size of installation on total cost per kilowatt.

mately \$45 per kw The total capital cost of the project, then, may be computed in accordance with Eq. 1, as shown in Fig. 2. At an average net head of 50 ft, 1 sec-ft will produce 3.47 kw (at over-all efficiency of about 82% from water to the step-up side of the transformers). On this basis, the power duration curve of Fig. 3 has been plotted from a long-term record of stream flows at the site.

On the basis of Fig. 3, the curve of Fig. 4 has been computed showing the average annual energy output, the average annual capacity factor, and the capacity factor during the week of minimum stream flow, all for various amounts of installation at the site of the Bridgeport hydroelectric project.

In Fig. 2 the decrease in total capital cost per kilowatt of capacity with an increase in installation is seen to be very marked for the Bridgeport hydro

project, which is more or less typical. With only 10,000 kw installed, the total capital cost is found to be \$645 per kw, whereas, if the installation is increased to 120,000 kw, the capital cost becomes \$95 per kw of capacity.

From Fig. 4 it is noted that the average annual energy production for 10,000-kw installation is 87,600,000 kw-hr (100% capacity factor). If the installation were ever increased to 120,000 kw, the possible average annual energy output would become 487,200,000 kw-hr at an annual capacity factor of 46.5%, and a capacity factor of 8.4% during the week of minimum stream flow.

Whether it will be economically advantageous to develop a project like the Bridgeport hydroelectric project will depend on (*a*) nearness to a load center, (*b*) shape and size of the load curve to which it might be connected, and (*c*) the annual cost of the project as compared to that of some other plant, steam or hydro, which might perform the same function. Section 19, Chapter 15, discusses the economic feasibility of this project under certain conditions.

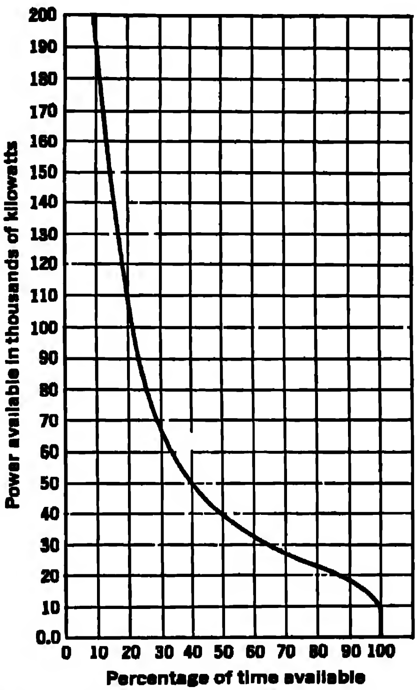

FIG. 3. Power duration curve for Bridgeport hydro project. 3.47 kw per sec-ft.

5. Transmission Cost. Steam plants are usually relatively near the load centers which they serve, although there are plants which, in order to secure an adequate source of condenser water and/or cheaper transportation for fuel, are located at some distance from the center of gravity of the loads which they serve. The steam plant engineer usually has considerable option in locating his plant to the greatest economic advantage. The hydroelectric engineer, on the other hand, does not have nearly as much freedom in his choice, as the location is largely determined by questions of topography, available water supply, and available head. Consequently, hydro plants are likely to be located far from the center of gravity of the loads to be served and frequently a transmission line of large capacity must be constructed for the delivery of substantially all the output to a load center. (See also Chapter 43, "Transmission Lines.")

For this reason the cost of transmission must usually be considered in connection with hydro plants. The cost varies greatly with the voltage utilized, the capacity of the line, and the degree of reliability required.

The data in Table 3, furnished by C. D. Gibbs,* may be useful in the preparation of rough preliminary estimates of the cost of transmission lines in connection with hydro projects. The costs given are more or less typical and do not include right of way, step-up or step-down substations, switching stations, or synchronous condensers when necessary.

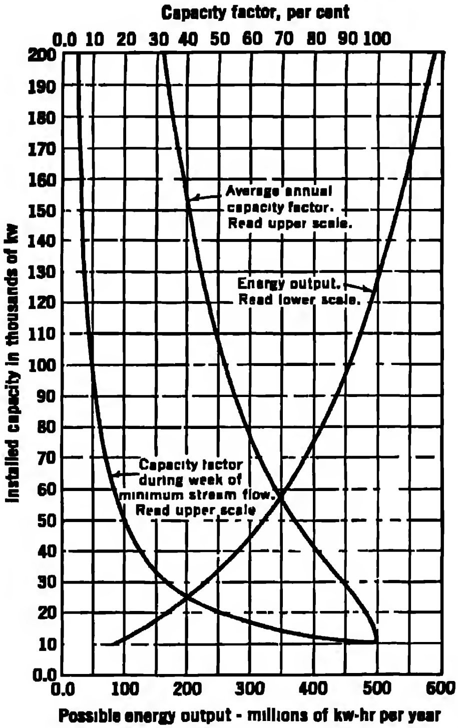

FIG. 4. Bridgeport hydro project. Average possible energy output. Average annual capacity factor. Capacity factor during week of minimum stream flow.

Prices are on the basis of labor and materials in 1939 and should be used only for rough preliminary estimates. Before a final preliminary estimate is presented, the transmission facilities required should be designed in some detail in accordance with Chapter 43, and a built-up estimate prepared.

Right-of-way costs for undeveloped territory may be less than $1,000 per mi, but in well-developed or suburban territory the cost of right of way for a high-tension line may exceed $15,000 per mi and in some cases has exceeded $100,000 per mi.

Switching stations without transformers (Chapter 42), where required, may cost $4 to $6 per kw of capacity. Step-up or step-down substations Chapter

* Electrical engineer, Day & Zimmerman, Inc., Philadelphia, Pa.

TABLE 3

APPROXIMATE CAPACITY AND COST OF TRANSMISSION LINES *

Description	Practicable Length for Stated Capacity, miles ‡	Conductor Size	Circuit Capacity, kw	Cost, dollars per mile †	
				Single Circuit	Double Circuit
33-kv wood-pole line	33	No. 4/0 B & S	10,300	7,500	10,000
66-kv H-frame wood-pole line	66	300,000 cm	29,600	10,000	....
66-kv steel-tower line §	66	300,000 cm	29,600	12,000	15,000
110-kv H-frame wood-pole line	110	400,000 cm	65,100	14,000	..
110-kv steel-tower line §	110	400,000 cm	65,100	15,000	18,000
154-kv steel-tower line	154	500,000 cm	114,000	20,000	..
220-kv steel-tower line	220	650,000 cm	212,000	23,000	..

* Capacity based upon a single circuit delivering power at the rated line voltage to a unity power factor load Under these conditions the line loss will be 8.66% of the delivered kilowatts At other than unity power factor the capacity will be in kilovolt-amperes, but the loss will be in kilowatts at the same percentage of delivered kilovolt-amperes. The loss will be directly proportional to the length for other distances.

† These costs based on prices in 1930.

‡ Other things being the same, the capacity for shorter distances will be approximately inversely proportional to the distance. Thus, if 10,000 kw is the capacity of a circuit 48 mi long, 40,000 kw will be the capacity of the same circuit 12 mi long.

§ It is rarely economical to use single-circuit steel towers for these voltages. Usually a two-circuit tower is installed, thereby providing for a future second circuit.

41) may cost from $8 to $14 per kw of capacity. Synchronous condensers, sometimes required on long high-tension lines up to 10 or 15% of the capacity of the line, may cost $15 to $20 per kw.

It is never satisfactory to have the output of a hydro plant dependent on a single transmission line. The arrangement should always be such that, if from any cause one transmission line goes out of service, the line or lines available can still deliver the full plant capacity. Otherwise the capacity could not be relied on It is much better, but not always practicable, to have

the transmission lines carrying power from a hydro plant follow different rights of way to the market.

If the capacity of a hydro plant were 100,000 kw one should have either two transmission lines of 100,000-kw capacity each or three transmission lines of 50,000 kw each.

The annual cost of transmission lines, including fixed charges and operation and maintenance, is usually taken at about 12% of capital cost.

6. Transmission Liability against Hydro Projects. A hydro project is sometimes spoken of as having a "transmission liability." By transmission liability is meant the additional annual cost that would have to be incurred for additional transmission facilities over that which would be incurred if, instead of hydro, steam were added to supply the requirements for increased capacity.

Not all hydroelectric projects have this transmission liability. Some territorial companies have located the large new steam plants on rivers where an ample supply of condensing water is available. Such locations are frequently quite remote from the center of gravity of the system load. In some instances where a hydroelectric project is being considered, it is thus found that the alternative steam plant would necessitate as large an annual cost for additional transmission facilities as the hydro project, and, consequently, in such a case there would be no transmission liability against the hydro.

There are some other cases where the transmission liability against the hydro is quite small because, although the required transmission facilities may be expensive, the proposed lines may be used for a dual purpose, as, for instance, interconnection between load centers to secure the advantages of diversity, or as one leg of a tie line to strengthen the connection between different parts of the system.

This question of transmission liability for any particular hydro project requires careful coordinated study by the electrical and hydraulic engineers and by the engineers charged with system planning. This study is of prime importance because frequently it determines the advisability of a given hydro project. If the hydro project is cheap enough so that it can bear the total cost of independent transmission facilities to the center of gravity of the load, the question is quite simple. However, for the reasons mentioned above, the actual transmission liability may be much less than the total annual cost of such transmission, and its determination in doubtful cases may require a great deal of study.

Under modern conditions the number of hydroelectric projects which are cheap enough to stand the cost of, say, 300 mi of transmission are not numerous if the only function of such transmission is to deliver the hydro output to market. On the other hand, if the transmission so provided forms a part of a network tying the load centers and plants of a great regional system together, the transmission liability against the hydro may be small and many such plants may be justified. For large hydro plants requiring 60 to 150 mi

of additional transmission for their sole use, the transmission liability may vary from $2.50 to $7.00 per kw per year of installed capacity.

7. Useful Life of Hydro Developments. A machine, a piece of equipment, or a structure has completed its useful life when annual maintenance and repairs become so high that, when considered in relation to the economies obtainable in a new up-to-date machine or structure, replacement is justified. Thus, wear and tear (in excess of ordinary maintenance), inadequacy, and obsolescence all play a part in determining the useful life of a machine or structure.

As an example of long life of an item of equipment, one of the authors cites a turbine which he recently had replaced. Installed in 1874 and in operation from then until 1937—63 years—the turbine was at first belted to a line shaft and later to a generator.

Since the first edition of this book was published additional evidence has accumulated as to the useful life of the various elements that go to make up a hydroelectric development. Most of this evidence indicates a longer useful life than had previously been assumed for many of the items. Some serious effort has been devoted to an attempt to remove the subject as much as possible from the realm of speculation. Investigations have been made into the life history of the various elements composing a number of developments. One of these investigations included the study of the life history of over 1000 items (structures and equipment) scattered over a wide area. The results of some of these investigations have been considered in the preparation of Table 4.

The useful life of the various items of a hydroelectric development recommended for application in computations to determine proper annual costs of depreciation (including obsolescence) were arrived at after due consideration to (a) the work of the public-service commissions; (b) investigations to determine useful life of actual units; (c) the recommendations of various authorities; (d) the personal observations of the authors

The useful life of any unit of equipment (or structure) must necessarily remain an estimate until that useful life is fully completed. All that the authors can say for Table 4 is that they believe the recommended figures on useful life to be conservative

8. Annual Depreciation for Hydro Plants. (See also Sections 4 and 5, Chapter 16.) In computing the annual cost of any hydro project or development, a determination of the annual cost of depreciation, in which obsolescence is included (as herein used), must be made. The sinking-fund method is recommended for engineering studies. It is assumed that each year a sum will be paid into a reserve such that at the end of the useful life of an item the reserve will be of sufficient size to replace it or its equivalent in usefulness.

Consequently, starting with the estimated useful life of an item, one would then determine the sum which it would be necessary to pay into the reserve each year in order to have that reserve equal the original cost at the end of

TABLE 4

USEFUL LIFE OF ELEMENTS OF HYDROELECTRIC DEVELOPMENT

	Estimated Useful Life, Years
Dams	
Earth dams	x
Mass concrete and masonry dams	x
Reinforced-concrete dams: arched dams, multiple-arch dams, flat slab buttressed dams	
(*a*) In mild climates	x
(*b*) In severe climates	20–30
Timber dams	40
Steel dams	30
Intakes	
Intake structures of concrete or masonry	x
Timber booms, untreated	15
Timber booms, impregnated above water line	25
Racks	20
Rack structures	40
Steel gates	35
Timber gates	15
Valves	25
Gate hoists, exposed	15
protected	30
Conduits	
Wooden flumes, untreated	15
impregnated	30
Steel flumes	25
Reinforced-concrete flumes	
(*a*) Mild climates	50
(*b*) Severe climates	20–30
Tunnels	x
Steel pipe, open	30
buried	20
Wood-stave pipe, untreated	20
impregnated	40
Concrete pipe	50
Powerhouse	
Concrete substructure of powerhouse	x
Brick and structural steel superstructure of powerhouse	50
Reinforced-concrete superstructure of powerhouse	50
Miscellaneous structural steel exposed to weather	35

TABLE 4—*Continued*

USEFUL LIFE OF ELEMENTS OF HYDROELECTRIC DEVELOPMENT

	Estimated Useful Life, Years
Powerhouse Mechanical Equipment	
Hydraulic turbines	30
Other mechanical powerhouse equipment including piping	30
Powerhouse Electrical Equipment	
Generators	25
Transformers	30
Circuit breakers	15
Lightning arresters	10
Other miscellaneous electrical equipment including wiring	20
Outdoor Substations	
Structures	40
Equipment	25
Transmission Lines	
With wood poles	20
With steel towers	50
Frame Dwellings	35

Items listed as x are believed to have an indefinite useful life if properly repaired and maintained.

that useful life. For instance, assume that a generator cost \$50,000. From Table 4, the estimated useful life to use for this purpose is found to be 25 years. Referring now to Fig 1, Chapter 12, or to Table 1, Chapter 16, it is found that, for each dollar of capital cost, it is necessary to pay into the reserve \$0.0209 in order to have the reserve, at 5% compound interest, amount to \$1 at the end of 25 years Thus the annual cost of depreciation for this generator would be \$50,000 × 0 0209 = \$1,045 00. (For accrued depreciation see Section 5, Chapter 16.)

The annual depreciation on hydro developments is relatively low. It is very much less than that on steam plants (from one tenth to four tenths as much). There are several reasons for the fact that annual depreciation on hydro is relatively small.

1. Items that have an indefinite useful life, such as land and riparian rights, earth dams, mass concrete dams, powerhouse substructure, tailrace, etc., usually comprise a very large part of the total investment.

2. The factor of obsolescence is small because the over-all efficiency already attained does not leave room for radical improvement. It is frequently found uneconomical even to rehabilitate plants of very early vintage.

3. The moving parts are few, and those that require the larger investment move at very low speed which, in itself, means long life.

For illustration, consider a certain hydro development which cost $200 per kw of capacity. Of this cost, $160 was for items having an indefinite life—lands and riparian rights, dams, and powerhouse substructure; $40 was for depreciable items—powerhouse superstructure, mechanical and electrical equipment, gates, etc. Assume that a study has indicated that the mean useful life of the depreciable items (from Table 4) is 30 years. From Table 1, Chapter 16, the annual deposit to accumulate $1 at 5% compound interest in a period of 30 years, is $0.0150. For the $40 worth of depreciable items, this would amount to $0.60 per year. As the remainder of the investment is in items having indefinite life, the annual depreciation per kilowatt of capacity would be $0.60 per year. The annual depreciation on the investment would in this example be $0.60 \div 200 = 0.3\%$.

The above example probably approaches the minimum depreciation rate that any hydroelectric development would have. If there were long steel penstocks and a long flow line conduit of wood-stave pipe, for instance, the depreciation would be much greater.

Probably the maximum rate of depreciation would occur for a project where no dam was involved and all the investment was in the powerhouse and equipment (as would be the case where a power company built a plant at a government dam). If this were the situation in the above example, the annual rate of depreciation would be $0.60 \div 40.00 = 1.5\%$. Consequently the range in the rate of annual depreciation for hydro projects may be said to be between 0.3 % and 1.5% of total capital cost.

9. Taxes and Insurance on Hydro Plants. The rate for taxes is generally lower for hydro plants than for steam plants because the rate of taxation is lower in the rural or backwoods sections where most hydroelectric plants are located than in the urban or industrial sections where most of the steam plants are located. Insurance also comprises a considerably smaller percentage of total investment with hydro than with steam, because the total investment in insurable items bears a smaller relation to the total cost.

In all comparisons of steam and hydro plants, it is well to eliminate for both all the items of total annual cost extraneous to the power plant itself, which cannot affect any decision that might be made. Thus, scarcely anyone would think it necessary or desirable to allocate to the annual cost of such steam and hydro plants any portion of the company's general and miscellaneous expense. Some engineers do, however, allocate to the plants a portion of income taxes and of gross receipts taxes, if paid. Although, if the allocation is fairly made, it does not vitiate the comparison, it is usually entirely unnecessary and merely introduces an additional complication into the computation and should therefore be avoided wherever possible.

An income tax is a tax on profits. Even though the construction of one plant of several alternatives may increase the profits and therefore the income tax of the company, this increase in income tax merely means that the company has to give up a part of the increased profit made. This fact would not keep one from developing the project from which the greater profit could be made. Consequently, the authors believe that it is better in comparative setups of this nature to eliminate any allocation of income taxes to generating plants.

It is practically never necessary to consider franchise and capital stock taxes. Some states collect a gross receipts tax from power companies. This is a tax on revenue. Revenue will not be affected whichever alternative plant is chosen for development. Consequently in studies of the sort discussed here it is usually unnecessary to consider such taxes.

Special cases may come up for which such taxes will have to be considered; for instance, one of the alternative plants might be located in a state which collects a gross receipts tax on the output of the plant, whereas at the other alternative project, located in another state, no such tax might be required. Also, there might be a license tax of so much per horsepower of installed capacity for developing water power but no corresponding tax for a steam plant.

In this book it is assumed, in the illustrative examples utilized, that it is necessary to consider merely those taxes directly applicable to the generating plants, which usually means merely property taxes. On this basis, taxes and insurance on hydroelectric plants will generally vary from about 0.5% to 1.5% of total capital cost. In the illustrative examples in this book, 1.0%, which is believed to be fairly typical, has been used for these items.

10. Cost of Money for Hydro Plants. (See also Section 5, Chapter 12.) In the financing of the capital cost of hydro plants, the annual cost of the money raised varies with the credit of the company and the condition of the money market. In the past, some hydroelectric projects have been undertaken by independent companies organized for the purpose, with the intention of wholesaling the power to various industries and public utilities. Sometimes such companies have had to pay a very high price for money. Ordinarily, however, the construction of hydroelectric projects is undertaken only by well-established public utilities, or else the enterprise is underwritten by a utility company having a sound credit rating. Consequently, the cost of money required for a hydro project is usually the same as that which the utility company has to pay for money to be used for other capital expenditures. Hence the cost of money for a hydroelectric project should usually be just the same as for a steam plant.

However, for any given case, the actual cost of money that will have to be paid should be determined as nearly as possible, and this rate should be utilized in the setup. This is frequently of material importance, because the amount of money required for a hydroelectric plant is usually much greater than that required for a steam plant of the same capacity. If the

rate chosen for the comparative setups is lower than the actual, it will tend to make the hydro look more favorable than it actually is, and vice versa.

As financing is usually handled, the cost of money includes the bond discount and expense, with the interest on bonds (covering about 50% of the cost), dividends on preferred stock (covering 20 to 30% of the cost), and the return on equity money the remainder.

For both steam and hydro plants, the actual total cost of money will in general vary from 5.5% to 8.5% per year of the total cost of the project. In illustrative cases discussed in this book, the cost of money has been taken at 7% per year of total capital cost, which is believed to be fairly typical under normal conditions for companies with sound credit (see also Section 7, Chapter 16).

11. Annual Fixed Charges on Hydro Plants. On the basis of the foregoing discussion, the total annual fixed charges on hydroelectric plants may be taken as in Table 5.

TABLE 5

TOTAL ANNUAL FIXED CHARGES ON HYDROELECTRIC PLANTS

	Usual Minimum Annual Rate, %	Usual Maximum Annual Rate, %	Typical Annual Rate as Used in Illustrative Cases in this Book, %
Total cost of money	5.5	8.5	7.0
Taxes and insurance	0.5	1.5	1.0
Depreciation and obsolescence	0.7	1.5	1.0
Total annual fixed charges (Percentage of total capital cost)	6.7	11.5	9.0

12. Annual Operation and Maintenance Cost at Hydro Plants. Annual operation and maintenance costs at hydro plants are more or less proportional to the capacity of the plant and the number of units. Such costs will vary also with the wage scale and the practices of different companies. Actual costs vary widely and are sometimes quite high per kilowatt of capacity for small plants Thus, there are some plants of 5000- to 10,000-kw capacity where annual operation and maintenance run from $4 to $6 per kw of installed capacity. There are other plants of the same size where the annual cost is less than $2 per kw.

When due consideration is given to the fact that small plants may be made automatic or may be remotely controlled, it is believed that, in general, for new plants, there is no need for having an annual cost for operation and maintenance (at the plant) in excess of $2 per kw even for plants as small as 5000-kw capacity.

For very large plants of 100,000- to 200,000-kw capacity, the annual cost of operation and maintenance may be as low as $.80 per kw of capacity.

This figure applies to new, modern plants with large-capacity units. Some of the largest hydro plants have been developed over a period of years and consequently contain units of varying size and age. Operation and maintenance in such plants are necessarily often very much higher than the figures given herein.

Although there is a considerable legitimate variation according to the location and type of plant, Table 6, based on an examination of records at a large number of plants, gives annual operation and maintenance costs, for hydro plants of various capacities, that may be taken as roughly typical for a large number of new plants in many sections of the country.

TABLE 6

ANNUAL OPERATION AND MAINTENANCE COST OF HYDRO PLANTS

Capacity of Plant, kw	Typical Probable Annual Cost of Operation and Maintenance at the Plant	Probable Cost of Operation and Maintenance per Kilowatt per Year
10,000	$ 18,000	$1.80
20,000	30 000	1.50
40,000	48,000	1.20
75,000	75,000	1.00
125,000	110,000	0.88
200,000	160,000	0.80

Such a list as that in Table 6 is useful only in making preliminary setups. As soon as the type of plant, number of units, and general arrangement of a plant are decided upon, a built-up estimate of this item of annual cost should be prepared. Figure 5 gives a curve based on the same data as the above list showing typical operation and maintenance costs at hydro plants of various capacities.

13. Total Annual Cost of Hydro Developments. The total annual cost of a hydroelectric development, unlike that of a steam plant, does not vary with the amount of energy turned out but remains practically the same from year to year. The total annual cost of a hydro development is the sum of the annual fixed charges (Section 11), the annual cost of operation and maintenance (Section 12), and the annual cost of the transmission liability (Section 6), if any.

Suppose that a certain hydro project is to have an installation of 100,000 kw * and is situated 30 mi from a load center. Under the existing conditions the only function which the transmission line will serve is to bring the power to market, and as it is feasible to install additional steam capacity right at the load center all the cost of transmission must be charged against the hydro project. Say that the cost of a double circuit line is $25,000 per mi,

* For convenience this is assumed to be the effective capacity of the plant at market after deducting losses.

making the cost of the transmission line $750,000, to which must be added $2,000,000 for cost of step-up and step-down substations, making $2,750,000. At an annual rate of 12% this will make the annual cost of transmission or "transmission liability" $330,000 per year or $3.30 per kw of capacity per year.

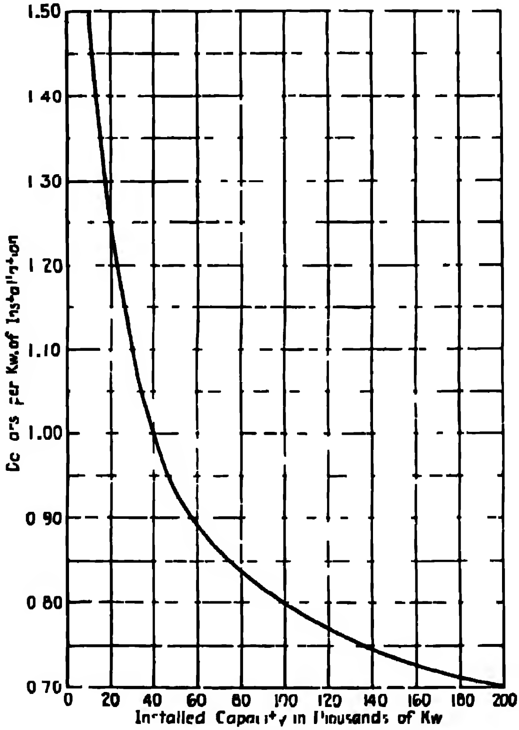

FIG 5 Operation plus maintenance costs at typical modern hydro plants (based on a study of records of over 30 plants) (From paper on "Economic Balance between Steam and Hydro Capacity" by K M. Irwin and Joel D Justin, *Trans A S M E*, Vol 55, No 3, p 63.)

The total annual cost of the hydro project per kilowatt of capacity with power delivered at market may then be computed as follows:

	PER KILOWATT
Annual fixed charges on hydro project (Section 11) 9.0% $145.00	$13.05
Annual operation and maintenance (Section 12)	0.93
Transmission liability (Section 6)	3.30
Total annual cost of hydro project per kilowatt delivered at market	$17.28

Whether the project would prove economically advisable will depend on the extent to which it can be utilized on the load curve, the amount of energy which it can produce, and the alternative cost of supplying the needs of the system by other means. These subjects are discussed in Chapters 14 and 15.

14. Total Cost of Power at Bridgeport Hydro Project. In order to provide a concrete illustration, use will again be made of the Bridgeport hydro project. Topographical surveys, subsurface investigations, and preliminary studies for this project have been completed. The site has been definitely determined and estimates made for the development of the site for various installations (see Section 4).

For the sake of simplicity it will be assumed that energy and power indicated in Fig. 4 are on a delivered basis. A thorough study of transmission liability has been made, and it has been determined that the transmission liability of the project will amount to approximately $3 per kw per year. Delivery, however, would not be at the same point for the various sizes of installation. Thus if the installation is 20 000 kw, delivery would be to one point, and if 160 000 kw, to another point.

Table 7 shows the manner of computing the total cost of power for a hydroelectric project delivered at a market where its cost can be compared with alternative sources of power from data already given herein.

TABLE 7

TOTAL COST OF POWER FOR BRIDGEPORT HYDROELECTRIC PLANT

(1) Installation kw	(2) Average Annual Capacity Factor (Fig. 4)	(3) Average Annual Energy Output (Fig. 4) millions of kilowatt hours	(4) Total Capital Cost (Fig. 2) millions of dollars	(5) Fixed Charges 9% of Col. 4 millions of dollars	(6) Total Annual Operation and Maintenance (Table 5) millions of dollars	(7) Transmission Liability of $3.00 per kw year millions of dollars	(8) Total Annual Cost at Market millions of dollars	(9) Cost per Kilowatt per Year	(10) Cost per Kilowatt hour
10 000	1.00	87.60	6.50	0.585	0.018	0.030	0.633	$63.30	$0.00723
20 000	0.963	170.00	7.00	0.630	0.030	0.060	0.720	36.00	0.00427
40 000	0.815	285.60	7.84	0.706	0.048	0.120	0.874	21.85	0.00306
60 000	0.682	358.00	8.76	0.788	0.063	0.180	1.031	17.18	0.00288
80 000	0.590	412.00	9.60	0.864	0.078	0.240	1.182	14.78	0.00286
100 000	0.521	454.00	10.50	0.945	0.092	0.300	1.337	13.37	0.00293
120 000	0.465	487.00	11.40	1.026	0.106	0.360	1.492	12.43	0.00305
140 000	0.422	517.00	12.30	1.107	0.120	0.420	1.647	11.76	0.00318
160 000	0.385	542.00	13.20	1.188	0.133	0.480	1.801	11.26	0.00334
180 000	0.358	564.50	14.10	1.269	0.146	0.540	1.955	10.86	0.00347
200 000	0.332	581.60	15.00	1.350	0.160	0.600	2.110	10.55	0.00363
250 000	0.283	615.90	17.25	1.553	0.193	0.750	2.496	9.97	0.00404
300 000	0.245	644.50	19.50	1.755	0.227	0.900	2.822	9.43	0.00439
350 000	0.218	667.50	21.75	1.958	0.259	1.050	3.267	9.35	0.00490
400 000	0.195	684.80	24.00	2.160	0.292	1.200	3.652	9.13	0.00545
450 000	0.177	696.40	26.25	2.363	0.324	1.350	4.037	8.96	0.00577

The figures appearing in Col. 3 are the average annual energy production delivered at market for the given installation. The assumption is made here that the load curve is of sufficient magnitude to absorb all the energy that the plant is capable of generating.

In Fig. 6 Bridgeport hydro project, the "total cost of delivered energy for various installations" is plotted from the data of Table 7. It will be noted in Fig. 5 that, starting with an installation of 10,000 kw (annual capacity

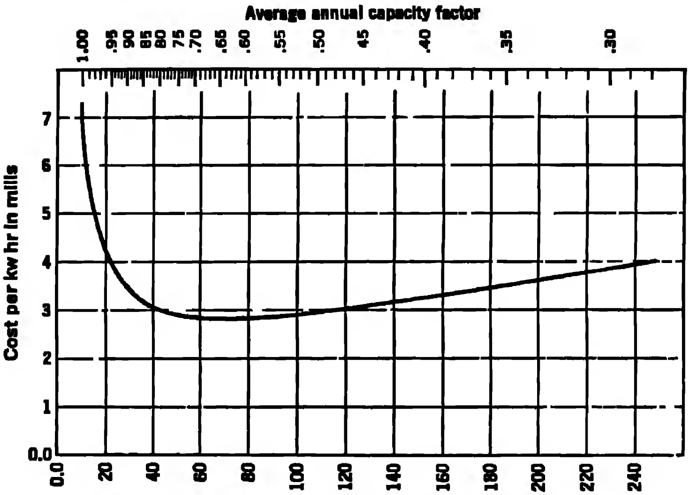

FIG. 6. Total cost of delivered energy from Bridgeport hydro project for various installations.

factor 100%), the total cost per kilowatt-hour decreases rapidly owing to the fact that a larger and larger percentage of the total annual stream flow is utilized by the additional installation (at relatively small incremental cost). Eventually, however, a point is reached where the additional energy obtained by each increment of installation decreases so rapidly that the total cost of energy starts to increase.

In any investigation to determine the feasibility of a hydro project it is desirable to plot curves like those of Figs. 2 to 5 inclusive. The determination of whether the project is economically feasible will be discussed in Chapter 15.

15. Bibliography. Below is a list of books and articles dealing in whole or in part with the cost of hydro power. In addition to the items listed here, see references under Bibliography of Chapter 12, "Cost of Steam Power," dealing with depreciation, obsolescence, and life of equipment.

1. H. K. BARROWS, *Water Power Engineering*, McGraw-Hill Book Co., New York, 1927.

2. JOEL D JUSTIN and WILLIAM G. MERVINE, *Power Supply Economics*, John Wiley & Sons, New York, 1934

3. H. K. BARROWS, "Hydro-Generated Energy (Cost of)," *Trans. A.S.C.E.*, Vol 104, p 1088, 1939

4 "Cost of Energy Generation," discussion by DANIEL W. MEAD, p. 1154; by WILLIAM E RUDOLPH, p 1164; by THEODORE B PARKER, p. 1183, in *Trans A S C E*, Vol 104, 1939

5 A C CLOGHER, "Hydro-Electric Practice in the United States," *Trans A S M E.*, Vol 59, No 2, p 65, 1937.

6 DANIEL W MEAD, "The Economics of Hydro-Electric Development," *Trans. A.S C.E*, Vol 86, p. 158, 1925

CHAPTER 14

MARKET REQUIREMENTS AND LOAD STUDIES

1. General. In order to determine whether an additional power plant is required, it is first necessary to know what the market requirements are and what they are likely to be during the next few years. This will require a knowledge of the following:

1. Magnitude of present load and its load factor.
2. Size, shape, and characteristics of the load curve for several years previous.
3. Existence of unconnected load, i.e., private plants in factories, office buildings, etc., and a knowledge of the economics of such private plants.
4. Probable growth of load over the next few years.
5. The present power supply system. Location, size, interconnection, and characteristics of existing power plants. This may affect market requirements as far as market for power from a proposed new plant is concerned.

2. Load Factor.* The load factor of any system is the ratio obtained by dividing the number of kilowatt-hours over a given length of time by the product of the peak load and the number of hours in this period. Thus we have daily load factors, weekly load factors, load factors for a given month, and yearly load factors. For instance, during a certain week a system turns out 8,400,000 kw-hr, and the peak load during the week is 100,000 kw. Therefore, the load factor for that week is

$$\frac{8{,}400{,}000}{100{,}000 \times 168} = 50\%$$

It is necessary to avoid confusing load factor (the ratio of average load to peak load) with capacity factor (the ratio of average load to installed capacity). For a discussion of capacity factor and its significance, see Section 8, Chapter 12.

The load factors of individual systems vary greatly according to the character of the load. A power system whose sales are almost entirely to a group of electrochemical plants may have an annual load factor in excess of 80%. In fact one of the largest power systems serving a highly industrialized section of the country which contains many electrochemical plants has an annual load factor in excess of 70%. Such high load factors are unusual, however.

* Where applicable, free use has been made of figures from *Power Supply Economics* by Joel D. Justin and William G. Mervine, John Wiley & Sons, New York.

In general a system serving a well-developed industrial territory will have an annual load factor of 40 to 60%, and a city without much manufacturing may have an annual load factor of 30 to 35%.

During the 1920's, when small systems were combining and when interconnections between systems were increasing, there was a rapid increase in annual system load factors. At present the increase has slowed down, but there is a slow long-term trend toward higher load factors, as additional and diverse uses for electric power are constantly developing.

3. Load Curves. Load curves are plotted using kilowatts as ordinates and time as abscissas. Many load curves are plotted on an hourly basis, the ordi-

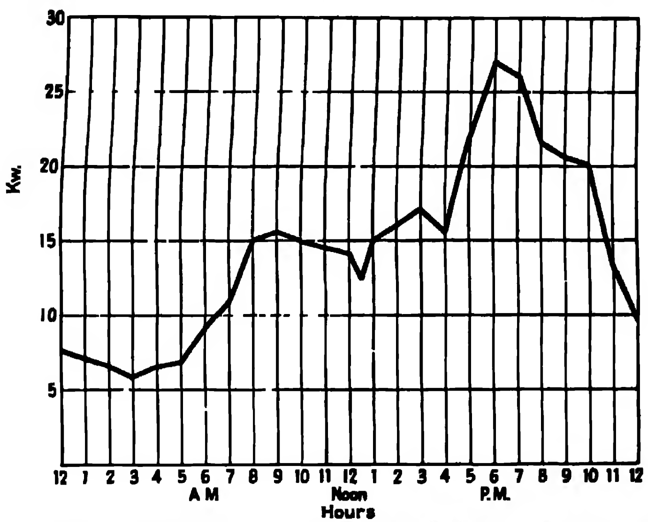

FIG. 1. December peak-day-load curve of a power system in which domestic and lighting use of electricity is predominating.

nates being simply the kilowatt-hours during each hour of the day. When using such load curves it is necessary to know the peak reached during the hours when the load is high, as this may in some cases be 15 to 25% greater than that indicated by the plotted curve.

It is becoming more and more common for power companies to plot their load curves using a 15- or 30-minute integrated peak. When the 15-minute integrated peak is used in plotting load curves, it is safe to apply the curves without alteration in making capacity studies, as the system will be able to carry the instantaneous peak over the 15-minute integrated peak represented on the diagram. This is because one can always get a little power beyond rated capacity out of the prime movers for a short time, although the efficiency on the increment of power is low.

Load curves are usually on a net generated basis and not on a sales basis. This is because the most convenient way to get the data is to obtain the

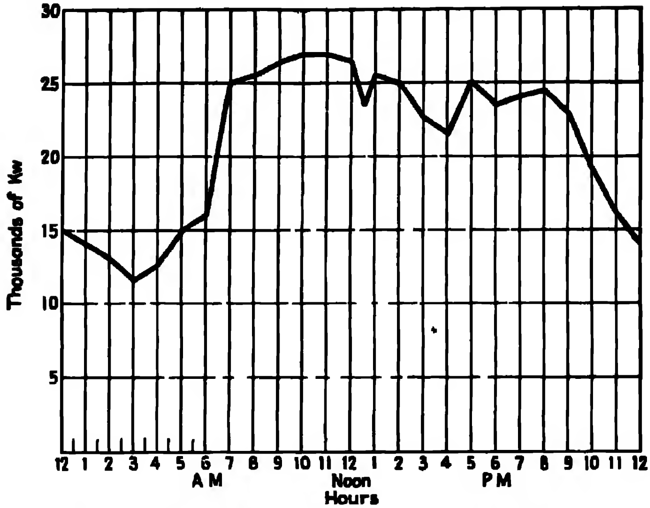

FIG. 2. December peak-day-load curve of a power system supplying chiefly industrial users.

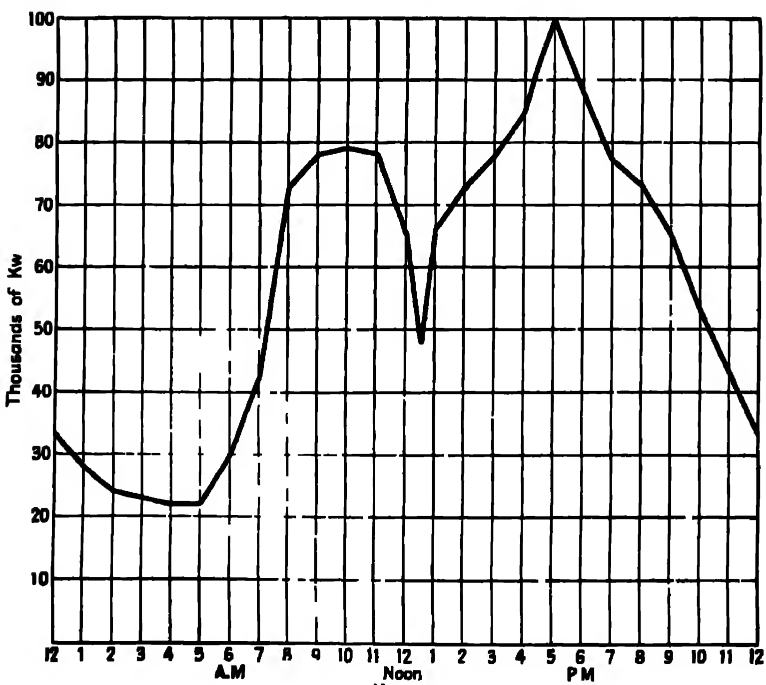

FIG. 3. Typical December peak-day-load curve of a metropolitan system load

records for the various power plants and then to total them up period by period However, if there are one or more hydro plants in the system which are remote from the load centers, the meter readings utilized are usually for a substation at or near the load center This practice puts the output of the hydro plant on approximately the same basis as that of the steam plants which are usually located at or near the load centers

For many systems the peak load of the year comes in November or December on some dark day during the early evening hours when the manufacturing load, the peaks for the commercial load, the electric railway load, and the lighting load all happen to coincide In some systems, however, the annual peak load occurs in September or October The load curve for the peak day of the year is usually the most critical single-day-load curve in connection with engineering studies for determining means of serving the loads, although there are often other days, weeks, and months which should be investigated, as they sometimes prove even more critical, especially in the case of hydro Figures 1, 2, and 3 show load curves for December peak-load days for systems serving loads of different characteristics

4. Relation of Monthly Peaks to Annual Peak Load. The summer peak load of most systems is materially lower than the peak loads for the fall and winter months Figure 4 shows a typical August peak-day-load

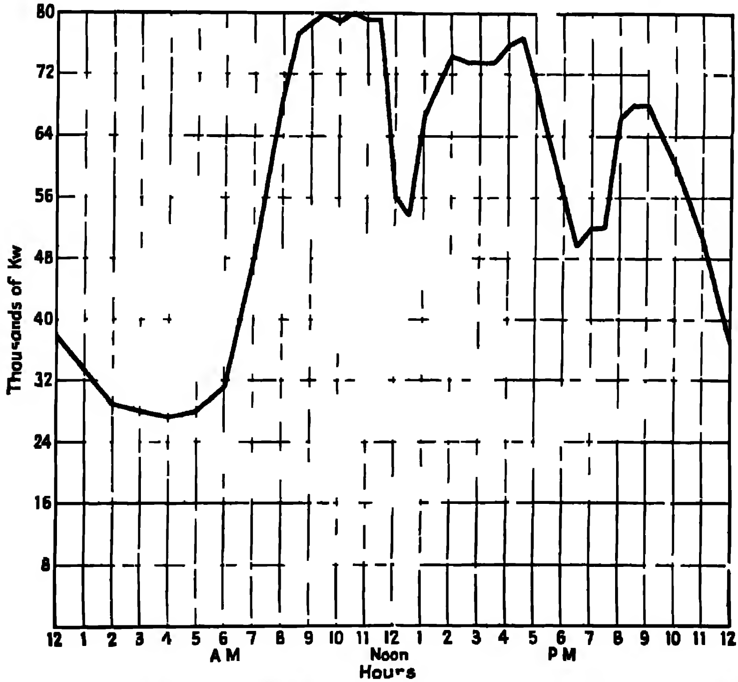

FIG. 4 Typical August peak-day-load curve of a metropolitan system load

curve of a metropolitan system. It is for the same system and the same year as the load curve shown in Fig. 3. Comparison of these two figures shows that the August peak load of this system is approximately 20% less than the December peak load. Table 1 gives the monthly peak loads of a system serving a typical industrial and metropolitan area, in terms of percentage of the annual peak load.

TABLE 1

MONTHLY PEAK LOADS AS A PERCENTAGE OF THE ANNUAL PEAK LOAD FOR A TYPICAL METROPOLITAN AND INDUSTRIAL POWER SYSTEM IN THE NORTHEASTERN STATES

	Monthly Peak as Percentage of Annual Peak
January	90.0
February	89.2
March	81.6
April	80.0
May	78.1
June	81.0
July	84.6
August	83.8
September	87.0
October	91.8
November	97.8
December	100.0

Although the relationship of monthly peaks to annual peak load is fairly constant, it changes somewhat in a fortuitous manner from year to year. There is also a definite slow long-term trend toward higher summer loads, due largely to the development of air conditioning.

It will be noted that during the period from April to August, inclusive, peak loads are in general from 15 to 20% less than the annual peak load, and this statement may be taken as applying generally to many systems. However, in some southern cities, summer loads are showing a tendency to increase materially owing to air conditioning, and it seems quite probable that in some parts of the country this tendency may affect to a considerable extent the relation of the monthly peak loads to one another.

The usual present monthly relation of the peak loads permits the "take-down schedule" to run from April to August, inclusive. During this period units are taken out of service, taken apart if necessary, checked, repaired, and adjusted. If such a low-load period were not available a system would need a greater amount of reserve.

5. Different Classes of Load. The various classes of load which a power system must serve are:

1. Commercial power—manufacturing, etc.
2. Railways and railroads—city traction and electrified railroads and interurban electric lines.

3. Commercial lighting—electric lighting of office buildings, stores, and factories and usually the power for elevators, pumps, etc., in such buildings.

4. Street lighting.

5. Domestic service—lighting and all household electric appliances.

6. Other utilities—usually this is power sold on a "when, as, and if" basis to other power companies, but it frequently includes sales of firm power as well

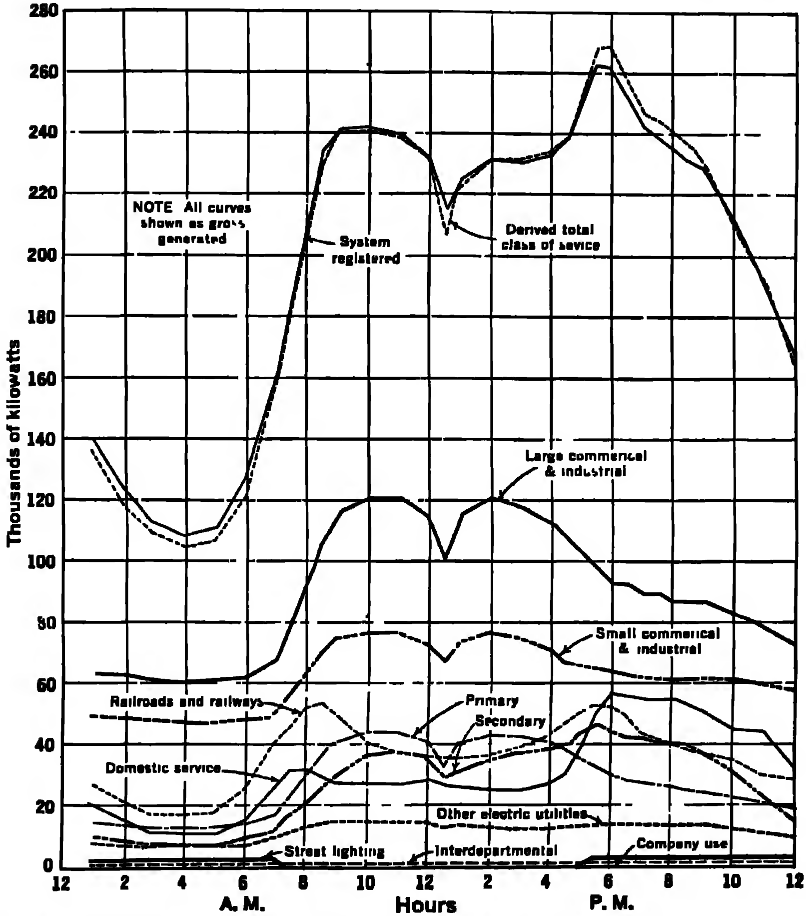

FIG 5. Load curve for peak December day, 1946, showing also component load curves for the various classes of service.

In Fig. 5 is shown a typical load curve for the peak December day, together with the load curves for the various classes of service listed above, which, added together, produce the load curve of the system as shown.

6. Weekly Load Curves. If one examines the weekly load curves for any given system for the past few years, he will be impressed by the simi-

larity of the shape of the weekly load curve for any given period in the year. Thus, if one were to take a typical weekly load curve for October of a recent year and multiply the ordinates by the growth ratio between the two periods, he would closely approximate the actual weekly load curve for October of the later year. This similarity in the shape of the weekly load curve for any given period of the year makes this curve particularly valuable in helping to determine how a proposed hydro plant may be fitted to the load curve.

7. Application of Weekly Load Curve to Hydro Studies. Weekly load curves are particularly useful in connection with capacity studies involving proposed or existing hydroelectric plants which have adequate pondage. Figure 6 shows such a weekly load curve for the week of maximum demand in December. The use of such a weekly load in connection with studies to determine the value of a proposed hydroelectric plant will now be explained.

In the system under consideration there is already in existence one hydro plant which has a capacity of 38,500 kw. This plant also has ample pondage for weekly regulation. In a week of minimum stream flow this plant can produce 450,000 kw-hr from the available flow in the river.

Measuring down from the 175,000-kw peak on the load curve, a distance is laid off equal to 38,500 kw (the capacity of the plant) A horizontal line is then drawn through this point, and it is found that, in the portion of the load curve thus intercepted, there will be 438,000 kw-hr during the week. Thus it is demonstrated that the entire capacity of the existing hydro plant can be used during this week to carry its portion of the load.

In other words the entire capacity of a hydro plant is "firm capacity" (see section 3, Chapter 15) and can be relied on even if, in the year of minimum flow, this minimum flow occurs at the time of peak load.

Let us say that the power company is considering the advisability of installing an additional hydro plant on another river. The head and other characteristics of the proposed plant are known, but the amount of the advisable installation has not been determined. The proposed plant will have pondage for weekly regulation and could produce from available stream flow 608,000 kw-hr in a week of minimum flow.

By trial-and-error methods, or preferably by means of a peak percentage curve (see Section 10 of this chapter), another horizontal line is drawn across the load curve in such a position that the portion of the load curve intercepted between it and the horizontal line previously mentioned which forms the lower boundary for the existing hydro plant contains the 608,000 kw-hr which the plant has available under the worst conceivable conditions of stream flow. By measuring the vertical distance between the two lines it is found that it corresponds to 13,000 kw as shown in Fig. 6. In other words, if 13,000 kw were installed in the proposed plant all of it would be useful or "firm capacity" under the most severe conditions conceivable.

If the load curve used were that predicted for the year in which the plant was to be completed, it is probable that something more than 13,000 kw

would be installed or else provision would be made in the construction of the plant to facilitate the installation of additional capacity. This is because, with growth of the load curve, the number of kilowatts intercepted between the horizontal lines would increase. Inasmuch as the value of firm capacity may be from $12 to $16 per kw per year and the incremental value of energy alone may be from 3 to 4 mills, the necessity for studies such as the above is quite evident.

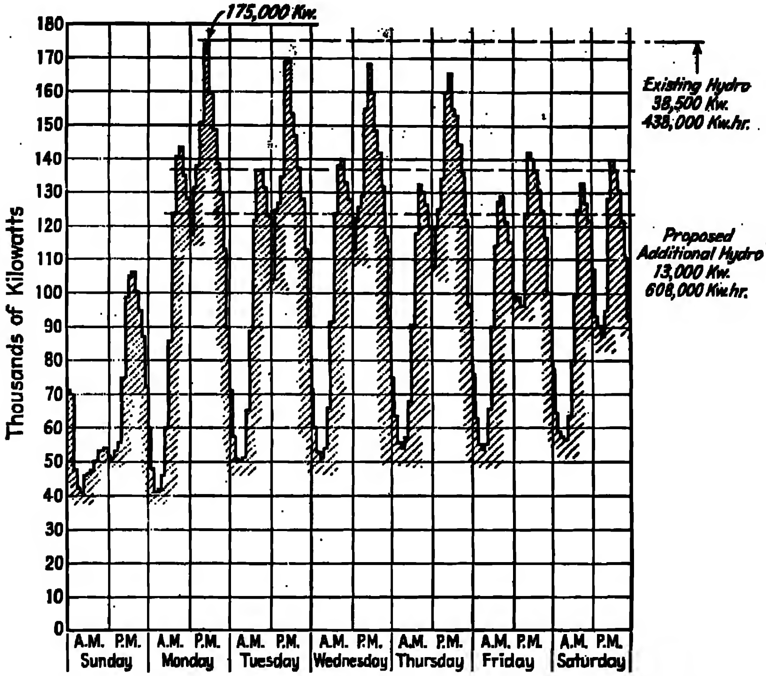

FIG. 6. Load curve, week of maximum demand in December 1937, Regional Power Company. Peak 175,000 kw. Energy for week 17,000 kw-hr. Load factor for week 57.8%.

In any actual case it would also be necessary to examine other weekly load curves because it may be that some other week, when the peaks are somewhat lower but longer, may be more critical. Also the authors have found one case where the minimum stream flow in September was so much less than it was in the peak-load month that a week in September became the critical period.

In Fig. 7 is shown the peak-week-load curve for one of the largest power systems in America. The system serves industrial cities, some of which have extensive electrochemical industries as well as contiguous small towns and agricultural districts.

The high base load results in an unusually high load factor of nearly 80%, but the general shape of the curve is not so very different from that in Fig 6 A more typical peak-week-load curve is shown in Fig 8 The system whose

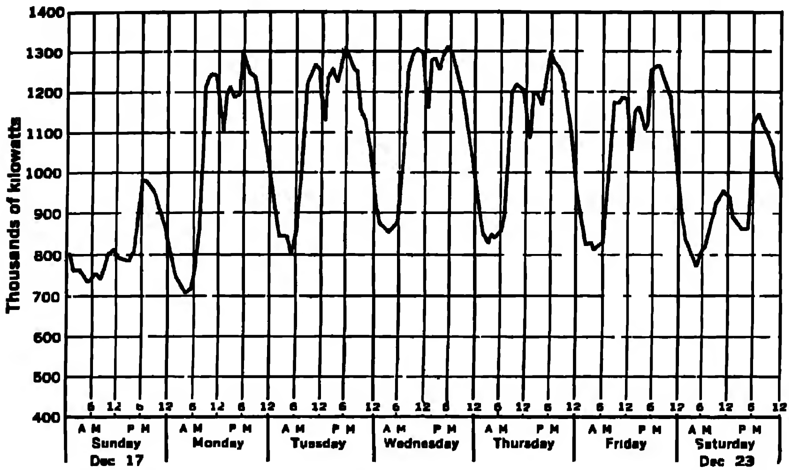

Fig 7 Peak-week-load curve of power system having high base load Load factor 78 6%

load is here represented serves the cities of Baltimore, Washington, and surrounding territory, as well as a portion of southeastern Pennsylvania, and is thus a composite of many different kinds of load

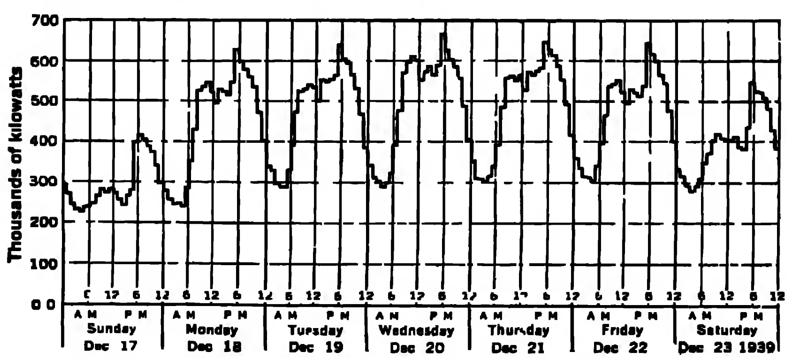

Fig 8 Hourly loads for peak week in December 1939 Combined Baltimore-Washington-Pennsylvania system Load factor 66 1%

8. Effect of Interconnection. Sometimes a proposed large hydro plant would serve not one system but several, thus obtaining effective interconnection between the systems served If there is diversity in load between the

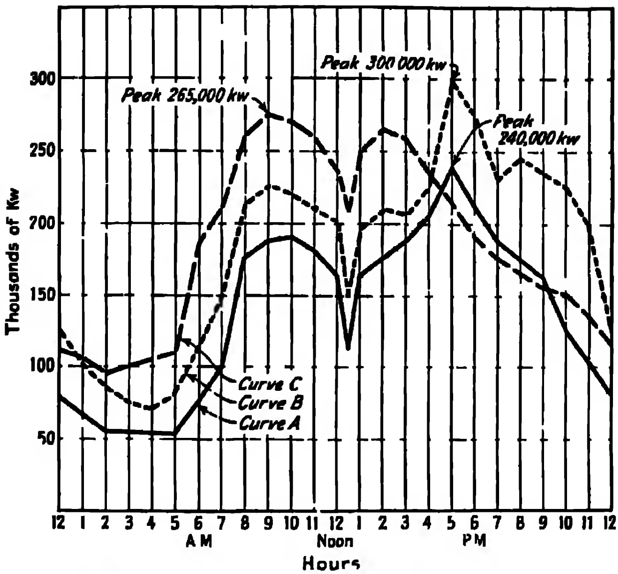

Fig. 9 Curves showing loads of three systems on peak day in peak month

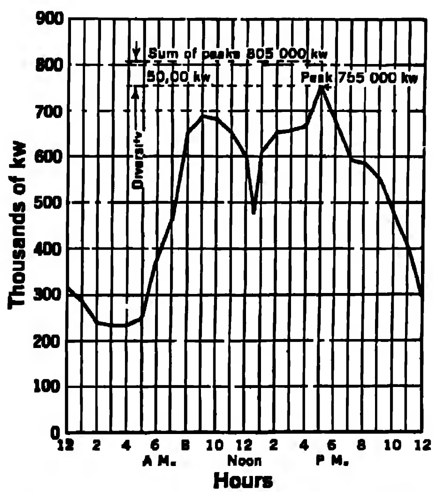

Fig. 10 Combined load curve for three systems shown in Fig. 9 showing diversity in load

systems, the combined peak load will be less than the arithmetic sum of the several peak loads. Thus in Fig. 9 is shown the load curve for three systems on the peak-load day of the peak-load month. These systems were then interconnected and this combined load curve for the same day is shown in Fig. 10 It will be noted that with interconnection the combined peak is 50,000 kw less than the arithmetic sum of the peaks of the three systems This, of course, is due to the fact that the peak of one company comes at a different time from that of the other two.

9. Load Duration Curves. If one arranges in order of magnitude the hourly loads carried by a system during any given week and then plots them

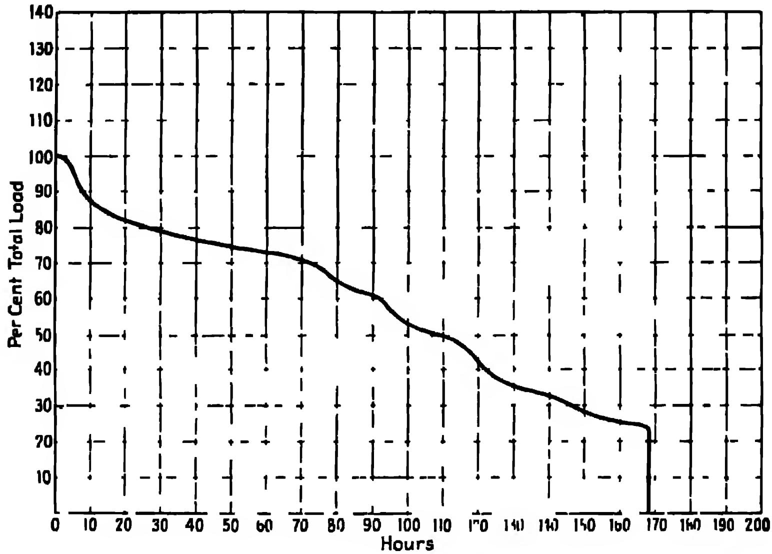

Fig 11 Load duration curve for peak-load week in December of large power company Load factor for the week 59 1%

on a diagram with loads as ordinates and hours as abscissas, he will have a load-duration curve for that week Any desired period of time, such as a week, month, or year, may be utilized. The typical week and the peak-load week for critical months are the most significant periods to use. For convenience in application to various years of the future, the load is usually plotted as a percentage of the peak load. Figure 11 gives a load-duration curve of a certain power company for the peak-load week in December.

This figure shows that the base load during the entire week was approximately 24% of the peak load during the week, that for 70 hr (for instance) out of the 168 it was equal to or greater than 71% of the peak load, and that for 20 hr out of the 168 it was equal to or greater than 82% of the peak load for the week. The area under the curve represents the total

energy for the week. Among other uses the load-duration curve may conveniently be used for allocating capacity. Hydro plants with pondage, or the older and less efficient steam plants, generally take care of the upper part of the curve, which has a low load factor. If, during the week in question, flow available at the hydro plant increases, its position on the curve would be lowered and the less efficient steam plant would take its place at the top of the curve.

10. Peak Percentage Curves. The peak percentage curve is derived from the duration load curve as follows: A number of horizontal lines (ten

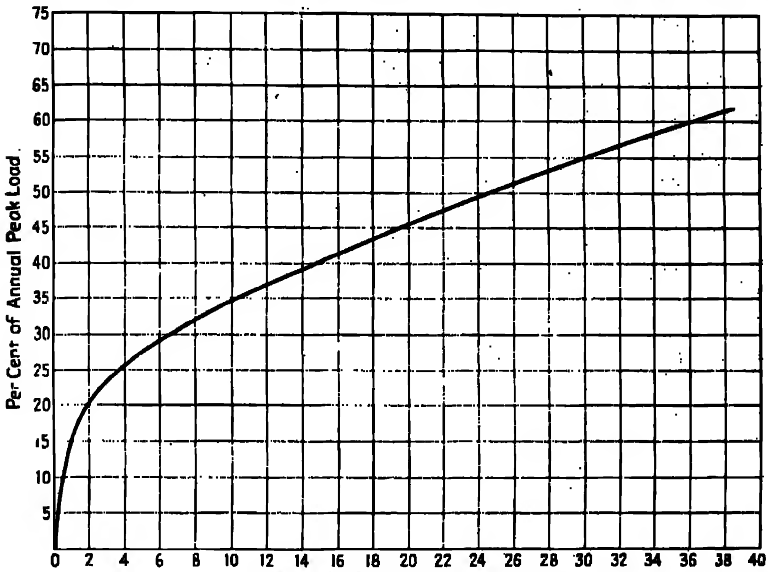

FIG. 12. Peak percentage curve for peak-load week in December of large power company. Load factor for the week 59.1%. Actual peak 492,000 kw. Total energy for the week 48,850,000 kw-hr.

are usually enough) are drawn across the diagram of the load-duration curve. Then by planimetering or some other method the total area below the curve down to each horizontal line is determined. From these data the peak percentage curve is plotted, usually with percentage of peak load measured down from the top as ordinates and percentage of total kilowatt-hours as abscissas. Figure 12 is a peak percentage for the peak-load week in December of a large power company and was derived from the load-duration curve of Fig. 11. The peak percentage curve of Fig. 12 indicates, for instance, that, in the top 25% of the load during that week, there is only about 3.8% of the total energy for that week.

To demonstrate further the use of such curves in connection with hydro studies, an illustrative example will be given.

Assumption. The power system* represented by Figs. 9 and 10 has at present its entire generating capacity in steam. Existing steam plants will soon be loaded up, and additional capacity must be provided. The advisability of adding hydro capacity instead of steam to the system is being considered, and an option on a site for a hydro plant has been secured. At the site it is feasible to develop 100 ft of head by means of a dam which will provide ample weekly pondage with slight drawdown. The river is a large one, but the minimum flow in December is so low that during such a period only 3,000,000 kw-hr per week will be available, although during a large part of the year there is a high flow in the river.

Question. What is the maximum size of installation which can be set up and which will be all firm capacity in a future year when the peak load has reached 700,000 kw and when the required total energy output of the system during the week of maximum December peak is 60,000,000 kw-hr?

This required energy output is determined by the fact that the known load factor of the system during the peak December week is 58.6%.

Solution. The hydro plant can be relied on to produce only 3,000,000 kw-hr during this critical period, which is (3,000,000 ÷ 69,000,000) = 4.35% of the total energy in the load curve for the week. By referring to Fig. 12, it is found that 4.35% of the energy (abscissas) is contained in the upper 26% of the load (ordinates). As the peak load is 700,000 kw, the greatest hydro capacity which could be installed and which would be all firm capacity in the year when the peak load has reached 700,000 kw is (0.26 × 700,000) = 182,000 kw.

From Fig. 11 and also from the annual load-duration curve (not here reproduced), it is found that this is just about equal to the base load of the system. Consequently, at times of high water practically all the hydro energy which this hydro plant could generate could be absorbed in the system. In other words, the proposed hydro plant would have a high utilization factor.

If the system considered had already had a number of hydro plants in existence, the principle would be the same but more computation would be required.

11. Reserve Capacity. Some years ago when a power system was often served by a single power plant, it was the usual practice to add one or more reserve units in a power plant. If a power plant required, say, four 20,000-kw units to serve the load (estimated for a few years after completion of the plant), it was customary to add a fifth unit and consider it the reserve unit. Today, when power systems are served by a multiplicity of power plants and are usually interconnected with other systems as well, this practice no longer prevails. All that is necessary for a power company to have always available is enough capacity to supply the anticipated peak load of the year

* This example is adopted from *Power Supply Economics*, Section 139, Joel D. Justin and William G. Mervine, John Wiley & Sons, New York.

ahead, with enough excess capacity so that it will be able to supply the load in case of any breakdown which it is reasonable to assume may occur.

Most companies, on the basis of their own experience and that of others, have adopted reserve criteria to which they attempt to adhere rather rigidly. Thus a suitable criterion for a large power company interconnected with other companies and with many power plants is that reserve capacity in excess of the next year's anticipated peak load shall be 10% of that year's predicted peak load or shall be equal to the capacity of the company's largest unit, whichever is greater. Other companies not so fortunately situated may require a greater ratio of reserve capacity, sometimes as high as 25%.

In addition, it is usual for the reserve criterion to require that at any given time the capacity which is hot and turning over shall be in excess of the load at that time by a margin equal to the capacity of the largest unit in service. Sometimes where continuity of service is excessively important, as in New York, criteria are even more severe than this.

12. Hydro Units as Reserve Capacity. It is necessary to have reserve capacity and to have some of it on the line at all times so that, if an operating unit fails, the load is immediately picked by unloaded or partially loaded units without any interruption in service. This factor is important when considering the advisability of adding a hydroelectric plant to the system.

Because the cost of keeping reserve steam plants hot and their turbines turning over so that they can pick up load in case of necessity is a heavy item of expense, hydroelectric plants have an inherent advantage for this sort of service. A hydraulic turbine with water in the scroll case and wicket gates closed can go from a stationary position to full load in 1 to 3 minutes, whereas it may require hours to fire up a cold steam plant and put it on the line. However, to function as the equivalent of "hot reserve," the hydro unit should be turning over.

A hydraulic turbine which is turning over under no load will pick up load almost instantly, whereas it may take 30 minutes to warm up a cold steam turbine and put it on the line.

As it costs practically nothing to keep hydraulic turbines ready to go on the line at an instant's notice, the advantage of having some hydro capacity in any power supply system is quite apparent.

Hydroelectric plants which have storage reservoirs are particularly adapted for this sort of service. Capacity equal to the required hot reserve which would otherwise be necessary can often be added in such plants at an economical cost, provided the connection of the hydro plant to the system through transmission lines is thorough enough so that this capacity can be relied on in an emergency. In some cases the operating cost on old steam plants maintained as reserve is high enough to more than cover the total annual cost including fixed charges on a peak-load hydro plant, possibly of the pumped storage variety, which might profitably perform the same function [4].

13. Limitations on Hydro Capacity as Reserve. There are limitations to the advisable extent to which hydro capacity should be relied on as reserve. Ordinarily one should be able to count on prime movers turning over and ready to serve, but not under load, as a part of the required reserve at any given time. In the case of a hydro plant remote from the load and dependent on a transmission line or lines over a single right of way, the entire plant should usually be considered as a single unit. That is, when such a hydro plant is operating fully loaded there should be available in the system hot reserve equal to the capacity of the plant. When only some of the units in the hydro plant are operating, none of the remaining units should be included in the "hot reserve" being relied on at that time. This is for the reason that a breakdown might take the form of a total interruption in the transmission lines (as by lightning), putting the entire plant out of service for minutes or hours. This would not apply if the plant were reached by more than one transmission line over different rights of way.

14. Load Prediction. It is usually only growth of load which necessitates the construction of power plants. So long as the installed capacity of a system will adequately serve the load with a fair margin for reserve, there is seldom any economic reason for installing additional capacity. There are, of course, rather rare cases in which, without growth, the greater efficiency of a new plant justifies its construction simply to supersede several old inefficient plants. In such cases it is necessary that the total annual charges of the proposed new plant, including fixed charges, be less than the mere operating cost of the plant or plants to be superseded.

Overly optimistic load predictions have been responsible for the construction of many plants which history has proved to be uneconomical for a number of years. Such mistakes put a financial burden on both consumers and owners which should be avoided as far as practicable.

At first thought it would seem a rather simple matter to predict load growth for a few years ahead. That it is not, however, the mistakes that have been made clearly show. Most of the load-growth estimates made in 1928 seem very strange now. In the decade of the 1920's, system peak loads were growing at the rate of 8 to 10% a year compounded, and load prediction seemed extremely simple. Although load prediction can never be very exact, careful study and investigation may be expected to remove many of the uncertainties.

When peak loads were growing constantly, load predictions were often made by simply extending the curve on the basis of the rate of growth during the previous few years. Of course, load predictions cannot be expected to foretell accurately the approach of depressions or booms. Figure 13 shows a load-prediction curve for a large system. The dot-and-dash extension represents a prediction made in 1927. The agreement with actual load was quite close up to 1929, but 5 years after that (1934) the actual load was nearly 200,000 kw less than the predicted. In 1933 a new load-prediction

curve was computed as shown by the dotted line. Actually realized peak loads, as shown by the full black line, later proved this prediction to be even less accurate than the first one. It might be mentioned that the system in question is among those which spare no pains in attempting to make predictions as exact as possible. The truth of the matter is that load predictions covering a period in excess of 2 or 3 years are nothing but guesses, as the unpredictable factors that may influence load growth are too numerous.

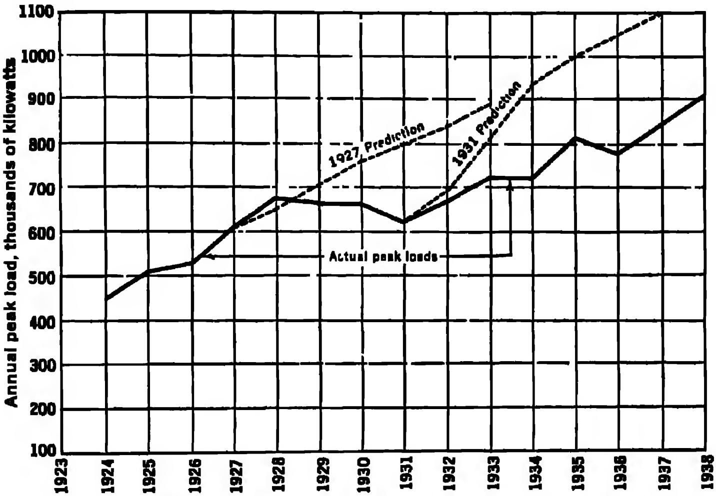

FIG. 13. Prediction of annual peak loads for a large power system.

Even a 2- or 3-year period of load prediction is surrounded by many uncertainties. Nevertheless it is essential that prediction should be made as accurately as possible because (especially for hydro plants) it may require that length of time to build and place a new power plant in service.

Just as is true of estimates of construction cost, a load-growth estimate which is broken down into component parts, with each part estimated as carefully as possible and then summed up, is likely to be more accurate than a load-prediction curve which is merely projected from past experience. The growth of each separate class of service should first be estimated by districts and the individual estimates combined to obtain the total load prediction.

In estimating the acquisition of new load, such as that of several factories not now connected, but with load which economic studies indicate can be acquired, it is necessary to realize that although a manufacturing plant has a motor installation of, say, 1000 kw, this installation will add much less than 1000 kw to the peak load of the system. In the first place, there is usually diversity in the use of the motors within the plant itself, and there will also be some diversity between this factory load and other factory loads. This

is a case in which the whole is practically always less than the sum of all its parts.

In a power system serving miscellaneous loads, the actual peak load is usually from ¼ to ⅓ of the total connected load.

The engineer charged with the responsibility of determining the advisability of installing an additional power plant in a system should secure the thorough cooperation of all those departments of the system in question which will help him to obtain the most accurate load prediction practicable, and should not accept without reservation the load-prediction estimates of the company's system planning division.*

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* A more detailed discussion of load prediction is given in Chapter 2 of Ref. 4 of the bibliography.

CHAPTER 15

CAPACITY OF THE DEVELOPMENT AND ECONOMIC ADVISABILITY OF HYDRO

1. General. Chapter 12 has considered the cost of steam power, which is generally the yardstick by which hydro is measured. Chapter 13 has been devoted to the cost of hydro power, and in Chapter 14 market requirements and load studies have been discussed. With this background we will now consider the capacity of any proposed development and the economic advisability of installing a hydro plant or plants.

2. Economic Hydro Ratio of a Power System. The hydro ratio is sometimes quite helpful as a rough indication of the probable advisability of additional hydro in situations where conditions are thoroughly understood. For any given power system served by both steam and hydro, there is an approximate economic balance between the proper amounts of steam and hydro installation. The hydro ratio, or ratio of the total hydro installation in the system to annual peak load, is sometimes used to express the existing relationship.

If the existing hydro ratio is in exact economic balance, then the total cost of power supply is at the minimum for the existing conditions.

The proper economic balance in any given situation between hydro and steam is largely dependent on the following factors: shape and magnitude of load curve (see Section 3, Chapter 14), availability and suitability of economical hydro sites, length of transmission required for hydro projects compared to steam ("transmission liability") (see Section 6, Chapter 13), runoff and its seasonal distribution for the stream considered, availability and cost of storage, cost of fuel, and availability of condensing water. The economic hydro ratio usually varies from 25 to 70%, but it may vary from zero, where no favorable hydro sites are available, to 100% in territories where there are hydro sites but where the cost of fuel is prohibitive.

Further increases in steam plant efficiencies and resulting decreases in the cost of steam-generated electric power will tend to decrease the economic hydro ratio. A general increase in construction costs or interest rates will have the same effect. On the other hand, further simplification in the design of hydro plants and/or a decrease in construction costs, or a lowering of interest rates, or an increase in the cost of fuel will tend to increase the economic hydro ratio.

3. Firm Capacity of Hydro Plants. The firm capacity of a hydroelectric plant can be defined as that portion of its total installed capacity

which can perform the same function on that portion of the load curve assigned to it as alternative steam capacity could perform.

Thus, the 13,000 kw of hydro capacity indicated in Fig. 6, Chapter 14, will be firm capacity if there is available during that week of maximum demand at least the 608,000 kw-hr of hydro energy required for it to fully function on its portion of the load curve.

Firm capacity is, therefore, dependent on the minimum stream flow at time of peak load, on the pondage available, on the installation provided, on the shape and size of the connected load curve, and on the interrelation of existing plants. Occasionally engineers speak of the minimum 24-hr power at a hydro plant as the firm power capacity of the plant. The two can be the same only where no pondage at all is available at the plant. With large pondage and favorable load conditions, firm capacity may be many times the minimum 24-hr power at the site.

The importance of firm capacity in its effect on the earning power of hydro plants is indicated in run-of-river plants without pondage. If such a plant, in addition to the liability of having no pondage, is subject also to the liability of being drowned out and thus becoming inoperative at the time of the system peak load, it then has no firm capacity. There must be just as much steam capacity in the system as though the hydro plant were not in existence, and all the plant accomplishes is to save fuel.

The firm capacity of a hydro plant may vary at different seasons of the year, but usually it is firm capacity at time of system peak which is of significance, and, unless otherwise specified, it is to be understood that the term "firm capacity" means the firm capacity of the hydro plant at time of system peak load. The peak percentage curve (see Section 10, Chapter 14) is a very useful tool for dealing with problems involving firm capacity.

The significance of firm capacity will be more fully understood by reference to Section 7, Chapter 14.

4. Capacity of the Development. Many factors enter into the determination of the most economical capacity for a hydroelectric project and into the determination of whether or not any development at all should be made. Most of these factors are so interrelated that a change in one affects many others. For important developments many studies must be made, each involving different assumptions of development arrangement, before a final scheme can be adopted and the economic advisability of the project determined.

The following factors are among those that should be considered in determining the capacity of a proposed hydroelectric project, its arrangement, and the advisability of its construction:

1. Market requirements (load curves), present and future (Chapter 14).
2. Available head (Chapter 9).
3. Natural stream flow (Chapters 3, 4, and 5).
4. Extent of possible regulation of stream flow (Chapter 10).

5. Cost of dam, land, and riparian rights (Chapter 13 and chapters on dams).

6. Incremental cost per kilowatt of power plant installation (Sections 2 and 3, Chapter 13, and Section 15, Chapter 15).

7. Firm capacity on load curve (Section 3, Chapter 15).

8. Capacity value applicable to the installation (Section 9, Chapter 15).

9. Energy value of the output (Section 8, Chapter 15).

10. Proposed installation (Chapter 15).

11. Gross head (Chapter 9).

12. Pond drawdown (Chapters 9 and 10).

13. Conduit losses (Chapter 8).

14. Tailrace losses (Chapter 9).

15. Productive net head (Chapter 9).

16. Pondage (Chapter 10).

17. Efficiency of equipment (Chapters 9, 38, and 41).

18. Cost of storage (Chapter 10).

19. Flow demand (Chapter 10).

20. Extent and method of regulation (Chapter 10).

21. Regulated stream flow (Chapter 10).

22. Governmental restrictions on regulation (required minimum release of flow) (Chapter 10).

23. Usable stream flow (Chapter 10).

24. Output (Chapters 9 and 10).

25. Portion of possible energy output which can be absorbed by load curve (Chapter 14).

26. Types of units (Chapter 38).

27. Advisable size of units (Chapter 38).

28. Other power plants, both steam and hydro, present and prospective (Sections 16 and 18, Chapter 15).

29. Cost of operation, maintenance, repairs, taxes, and depreciation (Sections 8, 9, and 12, Chapter 13).

30 Transmission liability (Section 6, Chapter 13).

31. Cost of money (Sections 10 and 13, Chapter 13).

Some of these factors, such as the market requirements (Item 1) and the natural stream flow (Item 3), should be readily determinable. The load curves for a series of years are practically always available, but estimates of future growth, on which the economic success of the project may be dependent, are sometimes too optimistic. (See Section 14, Chapter 14.)

Stream-flow records of some sort are generally available; nevertheless, inaccurate stream-flow records or the misinterpretation of accurate records has often led to too optimistic a forecast of the average annual output of the project. (See Chapters 4 and 5.)

Even such a relatively simple matter as the prediction of tailrace losses (Item 14) has sometimes been underestimated to such an extent that average annual energy production has been grossly overestimated. For the evaluation of all these factors and the determination of their relationships to one another, long experience in hydroelectric development is required. The mistake is occasionally made of intrusting the responsibility for development to those familiar only with the design of the structures involved and with the equipment required. The result is that many of the factors listed

above are certain to be neglected and the development will either be an economic failure or will produce a return lower than it would have if its conception, design, and construction had been placed in the hands of engineers competent in this field.

There is an old rule-of-thumb criterion to the effect that the installation in a hydroelectric project should not exceed that which could operate at full capacity for at least 30% of the time; i.e., stream flow equal to plant water capacity 30% of the time. This rule has no application for peak-load plants and probably it never was intended to have. However, it still serves fairly well as a rough preliminary guide to the probable amount of installation that will prove economically advisable for run-of-river plants with pondage under favorable load conditions.

It seldom pays to install a hydroelectric project unless nearly all the proposed capacity will be firm capacity on the connected load curve (see Section 3). This is because installation that is not firm capacity generally has very little earning value. For instance, consider Fig. 6, Chapter 14, which gives the load curve for the week of maximum demand in December for a certain power system. This system, as shown, already had one hydro plant with 38,500-kw capacity, all of which capacity is firm, for, if the minimum-stream-flow week coincides with this week of maximum demand, there will be at least 438,000 kw-hr of hydro energy available to supply the top 38,500 kw of the load. (The hydro plant is possessed of ample pondage.) Now then, it is proposed to install an additional hydro plant of 13,000-kw capacity with ample pondage, and the hydro energy available is not less than 608,000 kw-hr in a week of minimum stream flow. Consequently, as indicated by the load curve, this capacity would be firm and would have the same value as alternative steam capacity. If the cost of construction is not too high, it will probably pay to make the installation with provision for more installation as the load curve grows. During times of ample stream flow the hydro capacity would operate on the base of the load curve instead of the peak.

This criterion of firm capacity and its value, plus the value of energy produced, is a rough guide as to whether the project will prove feasible where other factors, such as cost of construction, are already known.

Under some conditions, however, particularly where there is a favorable market for secondary power, it pays to install a great deal more capacity than would be firm on the load curve. Some industries in which the cost of power is a material part of the total cost of the product, such as aluminum plants and paper mills, install excess capacity in the portions of their factories using the most power, as, for instance, pot rooms in the aluminum industry and pulp mills in the paper industry. They then make contracts with the power company for both primary power, which is furnished all the time, and secondary power, which is furnished "if, as, and when available" from water power. Naturally, the secondary power carries a much lower rate for

energy and no capacity charge per kilowatt or horsepower. Usually such secondary power is available from 4 to 8 months in the average year.

In some sections the market for such secondary power is an important consideration in determining the amount of installation for any given hydroelectric project and may justify a greater installation than would be firm capacity for some years in the future. The revenue from secondary power may provide a handsome return on the incremental cost of the additional installation.

5. Effect of Future Changes in Load Factor. The question of a changing load factor on the economic feasibility of hydro projects is often brought up in connection with purely peak-load hydro plants, which rely for their economic justification on the continuance of peak loads. Present system load factors are from 30 to 75% and there is a tendency for them to increase. A comparison of the load curve of a system having an unusually high load factor with one having a lower load factor shows that both curves have similar peaks and valleys and that the real difference between them is simply the greater base load of the system with the high load factor.*

This means that if we predict firm capacity for a hydro plant many years in the future, using present load curves as a basis, it may turn out that all the capacity will not be firm until some years later than predicted.

6. Hydro Plants as an Exclusive Source of Power. Some years ago many power systems in America were served exclusively by hydroelectric plants. To serve successfully a given load exclusively by hydroelectric plants, it is essential that the minimum flow of the stream (unless regulated) at time of maximum load be sufficient to furnish all the power required to meet that load.

Unless adequate storage was provided, there was, accordingly, a great waste of water over the dams for the greater portion of the year. Thus, minimum flow at time of peak load was the criterion for determining the permissible installation, with an extra unit or so for machine reserve. As a result, the total cost of the development was usually relatively high per unit of installed capacity. There are, however, exceptional streams, such as the Niagara, the St. Lawrence, and the Manistee, with flow so even that they are suitable for development as a sole source of power supply.

7. Effect of Interconnection on Feasibility. Interconnection has radically affected the economic relationship of steam and hydro power. It has produced large savings through diversity in load, reduction in necessary reserve capacity, diversity in construction programs, higher utilization factors on hydro plants, and higher capacity factors on the more efficient steam plants. Also, because of the resulting larger connected load, new capacity can be installed in larger and more efficient units and can be quickly loaded up.

8. Component Parts of Value of Hydro Power. The value of hydroelectric power can be divided into two components: energy value and firm-

* For instance, compare Figs 7 and 8 in Chapter 14.

capacity value, or merely capacity value as it is more generally called because there is no capacity value unless the capacity is firm capacity (Section 3). As previously shown (Chapter 12), steam is the yardstick by which the value of hydroelectric power and energy must be measured. The reason for dividing the value of hydro power into these two components is that each of them has its counterpart in steam-plant practice.

9. Capacity Value. As shown in Sections 12 and 13, Chapter 12, the annual total cost of any given steam plant may be divided into (1) fixed charges per kilowatt of capacity, (2) fixed cost of operating per kilowatt of capacity, (3) peak prepared for cost per kilowatt, and (4) variable cost of operating, usually stated in mills per kilowatt-hour. For preliminary comparative purposes, when an alternative hydro plant is being considered, the first three items are usually combined and designated the annual capacity cost, thus giving the annual capacity value for the alternative steam plant.

Whenever a capacity value per kilowatt is computed and applied to a proposed hydroelectric project, it is tacitly assumed that the alternative steam plant that would otherwise be constructed would have a capacity equal to the firm capacity of the proposed hydroelectric plant. Sometimes this assumption is unpracticable, and, if a steam plant were constructed, it might be either of very much greater or very much less capacity than the firm capacity of the proposed hydroelectric plant. In such cases, it is necessary to look farther ahead and make parallel computations of the total annual cost of power supply to the system for a number of years in the future for the construction of either alternative, as outlined in Sections 18 and 19 of this chapter.

Such computations are frequently desirable for precise determinations, but for preliminary setups it is usually sufficiently indicative to compute and utilize capacity value on the basis of the same amount of capacity for either alternative.

10. Energy Value. The unit value of the energy that a hydroelectric plant can produce from the available stream flow and that can be absorbed into the system is the same as the increment cost of energy (or variable cost of operating) that would obtain at an alternative steam plant. This statement is not always precisely true, however, and the exceptions are discussed in Sections 11 to 14.

11. Conditions under Which Value of Steam and Hydro Energy Are the Same. It has been assumed above that the value of the output of a hydroelectric plant would be determined by the annual capacity cost and the increment cost of energy at an alternative steam plant. For a system with several steam plants having various increment costs of energy, this assumption is precisely true if the hydro plant would operate at the same average annual capacity factor as the alternative steam plant. This would also be true if all the steam plants in the system were equally efficient, regardless of whether or not the proposed hydro plant had the same annual capacity factor as the alternative steam plant.

12. Conditions under Which Annual Capacity Factor of Hydro Remains Constant. A hydro plant, though subject to the more or less fortuitous variation in annual stream flow, operates over a term of years at the same average annual capacity factor and will continue doing so indefinitely, provided the load curve is large enough so that all the energy the plant can generate may be absorbed in the system.

13. Conditions under Which Annual Capacity Factor of Steam Plant Declines. On the other hand, a newly constructed steam plant is

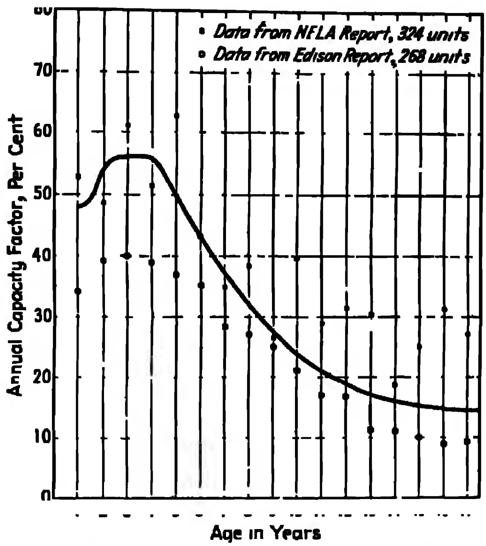

FIG. 1 Effect of age on annual capacity factor of steam plants.

usually the most efficient steam plant in the system and may be intended to operate on the base of the load curve at an annual capacity factor of perhaps 60%, displacing energy which would otherwise be generated at higher cost in the older steam plants. Figure 1, which is based on detailed operating records of a large number of steam turbine units, shows the effect which age has had on the annual capacity factors obtained in the past. The curve indicates that, on the basis of experience, one may expect a new steam plant to operate at or near an annual capacity factor of 50 to 60% for the first 5 or 6 years, after which the annual capacity factor may be expected to decline until it reaches about 16% by the fourteenth year of service.

14. Effect of Capacity Factor on Comparative Value of Hydro Energy. In view of the above discussion, it is believed that for preliminary comparative purposes an average annual capacity factor of 40% should be assumed for proposed steam plants when a hydroelectric project is being considered as a possible alternative. Then, if the proposed hydro plant had an

annual capacity factor of less than 40%, the hydro energy would be worth less than the increment cost of steam energy.

A setup showing total cost of power supply for a period of years in the future under the various alternatives is frequently necessary to determine the economic advisability of hydro (see also Sections 18 and 19).

15. Importance of Increment Cost of Hydro. The importance of the increment cost of hydro installation as a factor affecting the economics of hydroelectric plants has not always been fully realized. For instance, at a given site it was determined after investigation that a proposed 10,000-kw hydro plant could be constructed at a total cost of $3,000,000, or $300 per kw. This cost is high, and it will be assumed that additional steam capacity could be installed to furnish power and energy more cheaply.

The pond at this site would be of ample proportions, and it was found that if the capacity were doubled the total cost of the 20,000-kw project would be $3,700,000 (increment cost of installation, $70) (see Section 2, Chapter 13), giving a total unit cost of $185 per kw. The investigation showed that the 10,000-kw plant would be firm on the connected load curve and that 6 years hence the 20,000-kw plant would be firm capacity on the connected load curve. Also, owing to the fact that the larger plant would utilize a larger proportion of the total river flow, the 20,000-kw plant would have an annual output of energy about 25% in excess of the smaller plant. On this basis it was found that when the 20,000-kw plant would be firm capacity and hence could have credited to it full capacity value, it would be a cheaper source of power and energy that an alternative steam plant. Consequently, instead of there being an adverse report on the project, the site was purchased and held for future development.

A simplified setup for the two cases will now be given. In both instances, annual total cost of alternative steam power was $17.25 per kw for capacity and 2 mills per kw-hr for energy. The comparison assumes that there is no transmission liability against hydro as compared to steam.

CASE 1: 10,000-KW HYDRO

Capacity firm on load curve at time of installation. Note that alternative steam plant is identical with that represented by Curve A_2, Fig. 2.

Gross Annual Revenue on Basis of Steam Value

10,000 kw @ $17.25 (10,000 kw is firm capacity)		$172,500
50,000,000 kw-hr @ 2 mills (average year)		100,000
Total gross revenue creditable to hydro		$272,500
Deduct total annual cost of hydro		
Fixed charges @ 9% on $3,000,000	$270,000	
Operation and maintenance	15,000	$285,000
Net annual loss on hydro as compared to alternative steam		$ 12,500

CASE 2: 20,000-KW HYDRO

At same site as Case 1, to be built 6 years hence when the capacity will be firm on connected load curve.

Gross Annual Revenue on Basis of Steam Value

20,000 kw @ $17.25		$345,000
62,000,000 kw-hr @ 2 mills		124,000
Total gross annual revenue creditable to hydro		$469,000
Deduct total annual cost of hydro		
Fixed charges on $3,700,000 @ 9%	$333,000	
Operation and maintenance	25,000	$358,000
Net annual saving of hydro as compared to alternative steam		$111,000

16. Comparative Total Cost of Hydro and Steam Energy. The significance of the incremental cost of hydro installation (Sections 2 and 3, Chapter 13) in connection with the economic feasibility of a hydroelectric project becomes clear from a study of Fig. 2, which is a comparison between the total cost of energy from an actual hydro project with that for a proposed 400-lb pressure steam plant In Fig. 2 it is assumed that there is no transmission liability against the hydro plant. (See Section 6, Chapter 13.)

It is assumed also that the system can absorb all energy which the hydro plant may generate and that any capacity installed will be firm.

Ample pondage is available at the hydro plant, and it is noted from Curve C in Fig 2 that the capital cost of the hydro declines with increasing installation from about $565 per kw for 90% annual capacity factor to $100 per kw at 20% annual capacity factor.

Referring in Fig. 2 to the A curves, giving total cost of steam-generated energy per kilowatt-hour, and to Curve B, giving total cost of hydro energy per kilowatt-hour, it is seen that for very high annual capacity factors the steam plant shows lower unit cost of energy but that the hydro energy cost is less for all annual capacity factors lower than 70% even with a fuel cost of $3.00 per net ton (14,000 Btu coal).

Figure 3 shows curves of the same type as those in Fig. 2 indicating the comparative total cost of hydro and steam power for a particular hydro project and a proposed 12,000-lb pressure steam plant. The hydro is the Bridgeport hydro project,* and the steam plant is a highly efficient plant utilizing 1200-lb pressure.

The A curves of Fig. 3 are the same as those in Fig. 5, Chapter 12, and the data for the construction of Curves B and C are given in Table 7, Chapter 13.

Comparative cost curves like those of Figs. 2 and 3 of this chapter are useful in the study of the economic possibilities of any hydro project but are

* See Sections 4 and 14, of Chapter 13, and Section 18 of this chapter for a discussion of this project.

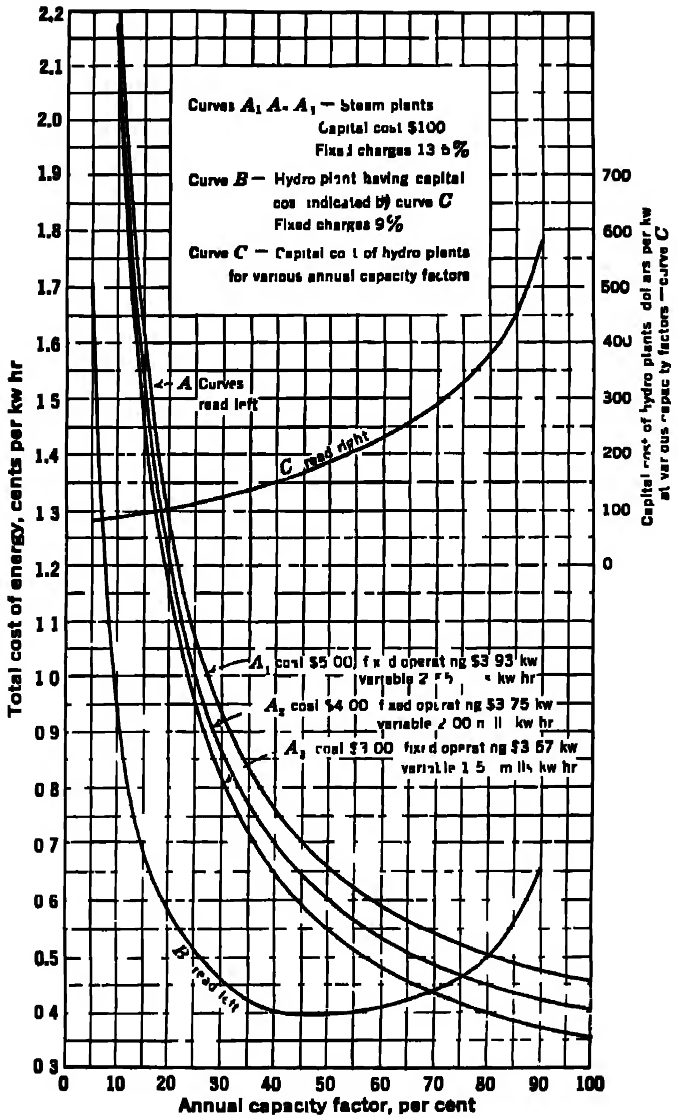

FIG 2 Total cost of energy at a certain hydro project compared to that at 400-lb pressure steam plant (no transmission liability assumed)

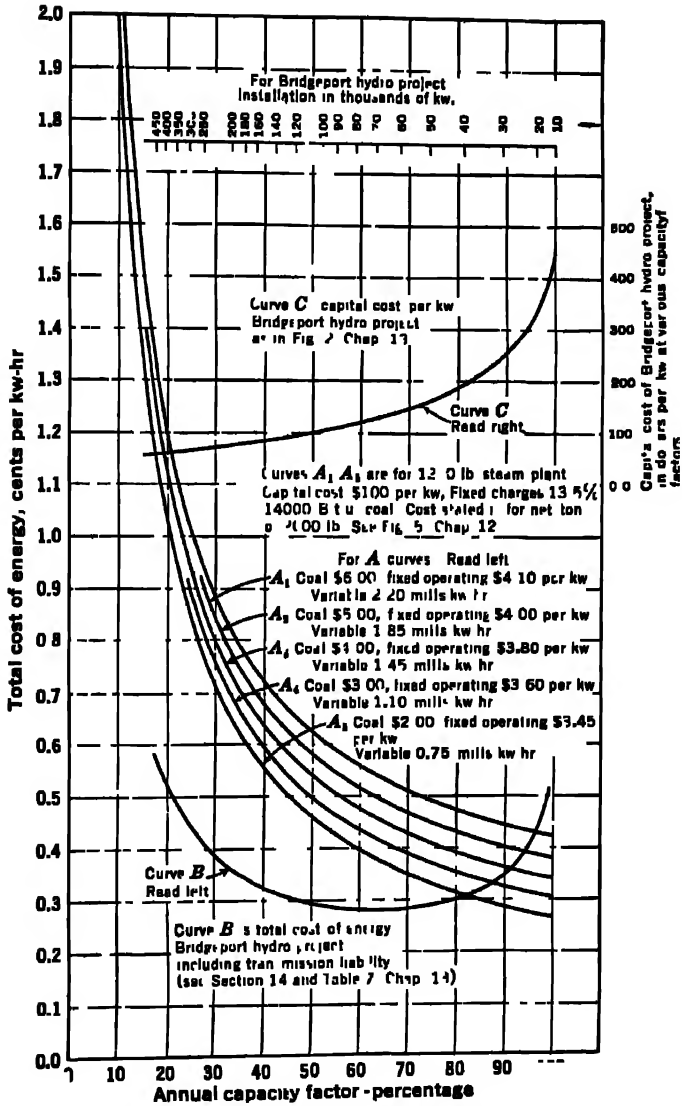

FIG 3 Total cost of energy at Bridgeport hydroelectric project compared to that at 1200-lb pressure steam plant

not, in themselves, conclusive as to the economic feasibility of the hydro project, unless it is known that all the proposed capacity will be firm, that all the possible energy output may be absorbed in the system, and that transmission liability has been considered. Also, the comparative value of the hydro may be decreased if the conditions within the system are such that the steam plant can produce economies by operating on a higher annual capacity factor than the hydro is capable of. Sections 11, 12, 13, 14, and 19 discuss this matter in detail, and Section 20 gives a comparative economic setup that takes these factors into account.

17. Economic Analysis of a Typical Hydro Project. It will be assumed that for a certain hydro project thorough field investigations, office studies, and estimates have been made, and it has been determined that 100,000 kw of capacity can be installed at the site and be firm capacity on the connected load curve in the year that the project will be completed. The available pondage is ample for weekly regulation with only slight drawdown. The minimum amount of energy available during a peak-load week has been determined as 4,000,000 kw-hr, which means that during such a week the plant would operate at a capacity factor of 23.8%. The feasible output of the plant in the average year has been determined as 480,000,000 kw-hr, giving an average annual capacity factor of 55%.

The growth of load in the system is such that about 100,000 kw of additional capacity will be required in the year in which the proposed hydro project can be completed. Although not more than 100,000 kw of capacity at this hydro project could be firm at the time the project is completed, it is evident that the characteristics of the project make it suitable for additional installation some time in the future, and, accordingly, provision has been made in the design so that this may be readily done. The total capital cost of the project has been estimated at $17,500,000, exclusive of transmission.

A careful study has been made of the transmission liability of the project (see Section 6, Chapter 13), and it has been determined that, for the project to be comparable to the alternative steam plant, it will be necessary to charge against the hydro plant the cost of transmitting its output to a secondary load center 60 miles from the plant, from which point the output will be distributed in various directions over existing facilities. The cost of the additional transmission facilities required which (although they may have some future additional function) must be charged against the hydro project has been estimated as $3,100,000. Taking total annual cost on transmission as 15% gives an annual transmission charge of $465,000, so that the transmission liability against the project is $4.65 per kw per year of installed capacity.

The alternative steam plant of 400-lb pressure, using 14,000 Btu coal at $4.00 per net ton, would be located at or near the above secondary load center, and its annual cost has been determined as $17.25 per kw per year

for capacity (fixed charges plus fixed cost of operating) and 2 mills per kw-hr for energy.

As the losses in transmission have been determined as 6% on capacity at time of full demand and an average of 4% in energy, the hydro power delivered would be $0.94 \times 100{,}000 = 94{,}000$ kw, and the hydro energy delivered would be $0.96 \times 480{,}000{,}000 = 460{,}800{,}000$ kw-hr.

Based on the above data and assumptions, a setup for the project can now be made.

A. Annual value of hydro power delivered in average year (based on costs at alternative steam plant)

94,000 kw delivered @ $17.25	$1,621,500
460,800,000 kw-hr delivered @ 2 mills	921,000
Total annual value of hydro power	$2,542,500

B. Annual total cost of hydro power delivered *

Fixed charges on hydro development	
9% on $17,500,000	$1,575,000
Operation and maintenance of hydro plant	80,000
Total annual cost of transmission (see above)	465,000
Total annual cost of hydro power delivered	$2,120,000

* See Section 13, Chapter 13.

Hence, in this example, the annual advantage of the hydro plant over the alternative steam plant is $2,542,500 − $2,120,000 = $422,500 per year. This would usually be considered quite a satisfactory margin inasmuch as additional capacity can be added quite cheaply when required. The hydro plant discussed is not an unusually cheap project, as the total delivered cost is ($17,500,000 + $3,100,000) ÷ 94,000 kw = $220 per kw.

It is believed that the above method is preferable to any attempt to figure "the return on the investment" (which is necessarily a more or less theoretical figure), because it answers the question that really interests the power company, namely, "Which source of power supply is the cheaper under the given conditions?" When the return on the investment is required, it is sometimes obtained by using the present average cost of steam power as the criterion. On this basis, the return on the investment for a case like the above often works out to a very high figure, but it proves nothing because a new, more efficient steam plant might produce an even greater saving.

Quite frequently, a public-utility executive charged with making the decision whether to build the hydro plant discussed above would look at the problem in the following manner: $9,400,000 must be spent for a steam plant to take care of the increasing load. If, however, a hydro plant is built instead, the required capital expenditure would be $20,600,000 (including

transmission), or $11,200,000 more than required for a steam plant. What return will the power company get on this additional investment?

This question can be answered as follows, from the data already given:

ANNUAL COST OF DELIVERED HYDRO POWER EXCLUDING INTEREST ON EXCESS INVESTMENT IN HYDRO

Depreciation, taxes, and insurance on hydro development (Sections 8 and 9, Chapter 13), 2% on $17,500,000	$ 350,000
Operation and maintenance on hydro plant (Section 12, Chapter 13)	80,000
Operation, maintenance, depreciation, taxes, and insurance on transmission facilities, 8% on $3,100,000 (Section 5, Chapter 3)	248,000
Interest charges on cost of alternative steam plant, 7% on $9,400,000	658,000
Total annual cost of delivered hydro exclusive of interest on excess investment of $11,200,000	$1,336,000
Annual sum available for return on excess investment in hydro $2,542,000 (annual cost of alternative steam plant) minus $1,336,000 = $1,206,500	$1,206,500

Return on excess investment in hydro $1,206,500 ÷ 11,200,000 = 10.7%.

As this rate is considerably in excess of the average net return on public-utility property, it indicates the advisability of the investment in the hydro project.

18. Estimates of Total Annual Cost of Power Supply. In many cases, in order to determine the most economical increment of capacity, it is desirable to estimate the total annual cost of power supply of the system for 5 to 10 years in the future, omitting from consideration all fixed charges on capital investment already made, but including all fixed charges on additional investment. Two complete sets of computations should be made, one for the case in which the steam plant is added, and one for that in which the hydro plant is added. Sometimes several optional programs are involved, representing as many sets of computations for various combinations of proposed plants. Such a study should be projected to the time when the alternative steam plant with the largest proposed installation will become fully loaded, as it sometimes happens that a large steam plant produces large operating savings, the effect of which is not entirely evident until the plant is loaded up.

In such cases, it is necessary to work out load-duration curves (see Section 9, Chapter 14) for typical weeks of each month of the years to be considered, and then to allocate the existing and proposed plants to serve the load curve most efficiently. Although future loads and actual allocation of plants to the loads may turn out to be quite different from those assumed, computations, if properly made, will show very conclusively the relative

total annual cost of power supply each year under the various options. The scheme that on the average shows the minimum total annual cost of power supply is the one that should be adopted. (See also Sections 16 and 17 of this chapter.) Such a precise study will sometimes produce results at variance with the conclusion reached from preliminary investigations. A study of this type is discussed in the following section.

19. Economic Advisability of Bridgeport Hydro Project. A certain power company owns the site of the Bridgeport hydro project (see Sections 4 and 14 of Chapter 13), and as its load is increasing rapidly the power company is considering the advisability of installing additional capacity. A careful estimate of load growth (see Section 14, Chapter 14) shows that, for 1950, the peak load of the system was estimated as 1,100,000 kw and that in 1949 additional capacity was required in order to maintain the proper amount of reserve (see Section 11, Chapter 14). The power supply of the system was entirely from steam plants, and a site had been obtained for a new steam plant on a river near the load center where ample water supply was available for condenser water, and preliminary studies of its design and cost had been completed. The proposed steam plant of 180,000-kw capacity would be a high-pressure plant (1200 lb pressure) utilizing 14,000 Btu coal costing \$4 per net ton. The total cost of energy output at this plant is given by Fig. 3, this chapter, and by Fig. 5, Chapter 12.

It must now be determined whether a hydro plant of 180,000-kw capacity at the Bridgeport site would perform the same function as the proposed steam plant and also whether there would be any net annual saving if the hydro plant were constructed instead of the proposed steam plant.

The critical period is the peak-load week of December. If the week of minimum stream flow should coincide with this peak-load week, the capacity factor of the Bridgeport hydro with 180,000-kw installation would be 5% during that week (from Fig. 1, Chapter 14). That is, the Bridgeport hydro may be relied on during such a week to produce $180{,}000 \times 0.05 \times 168 = 1{,}512{,}000$ kw-hr, which, because of the adequate weekly pondage available, may be distributed throughout the week in any desired manner.

The system load factor for the peak-load week has been found to be 65%. Thus, the total energy that must be generated in the system during the peak-load week of 1950 would be $1{,}100{,}000 \times 0.65 \times 168 = 120{,}200{,}000$ kw-hr. The ratio of hydro capacity to peak load is $180{,}000 \div 1{,}100{,}000 = 0.1635$.

It has been found that the peak-load percentage curve for the system (see Section 10, Chapter 14) is the same as that shown in Fig. 12, Chapter 14. From this figure it is found that, in the top 16.35% of the load curve for that week, there was only 1.1% of the total energy for the week, or in this case $120{,}200{,}000 \times 0.011 = 1{,}322{,}200$ kw-hr.

Inasmuch as it has been found above that 1,512,000 kw-hr is available at the Bridgeport hydroelectric project in a week of minimum stream flow, it is evident that the entire capacity of 180,000 kw can be relied on as firm capac-

ity (Section 3, Chapter 15). In such a week the plant would be operated to clip the peaks off the top of the load curve. The remainder of the load, or (1,100,000 − 180,000) 920,000 kw, would be carried by the steam plants of the system. This is the most severe condition possible but one for which the system must be prepared.

The next question is whether all the possible annual output of the hydro plant can be absorbed within the load (565,000,000 kw-hr in the average year as per Fig. 4, Chapter 13. To determine this it is necessary merely to examine the annual load-duration curve of the system (not here reproduced) and to know the requirements of the system. From the annual load-duration curve it is found that the 100% time base load of the system is 275,000 kw, and it is also known that in order to be ready to take variations in load it is necessary to have at least 70,000 kw of steam capacity on the base of the load curve at all times. Consequently, it is evident that all the energy that a 180,000-kw hydro plant may generate with plenty of stream flow available can always be absorbed in the system.

The average annual energy output of the Bridgeport hydro project (180,000 kw) will be 565,000,000 kw-hr (from Fig. 4, Chapter 13), giving an annual capacity factor of 35.8%. On the other hand, the alternative steam plant would produce a good deal more energy because it will be the most economical steam plant in the system. For some years (see Section 13) it will operate largely on the base of the load curve at a relatively high annual capacity factor estimated as averaging 65% (until additional capacity is installed). The average annual energy production of this alternative steam plant will thus be $180{,}000 \times 0.65 \times 8760 = 1{,}025{,}000{,}000$ kw-hr. As the annual load factor of the system is 45%, the total energy production of the system for 1950 will be $1{,}100{,}000 \times 0.45 \times 8760 = 4{,}340{,}000{,}000$ kw-hr. The variable operating cost (see Section 12, Chapter 12) at the alternative steam plant is 1.45 mills per kw-hr (from Fig. 5, Chapter 12), whereas for the existing steam plants in the system this variable cost of operating is 2.15 mills per kw-hr, or 0.70 mill more than that for the proposed steam plant. As will later be evident, this difference in the variable cost of operating is an important factor in considering the advisability of undertaking the construction of the Bridgeport hydroelectric project.

With the data above and those on the alternative steam plant (Fig. 5, Chapter 12; Sections 3 and 14, Fig. 2, and Table 7 of Chapter 13),* alternative setups can now be made for the year 1950 showing the total annual cost of power supply for the system if the alternative steam plant is installed or if the Bridgeport hydroelectric project is developed.

Table 1 shows these alternative setups. It will be noted that a net annual saving of $1,495,000 is indicated in favor of the construction of the Bridgeport hydroelectric project.

* The desired data may also be obtained from Fig. 3 of this chapter.

TABLE 1

Total Annual Cost of Power Supply for the Year 1950 under Alternative *A* (180,000-kw Steam Plant Added to System) and Alternative *B* (Bridgeport Hydro Project 180,000-kw Constructed)

Peak load 1,100,000 kw, total annual energy requirements 4,340,000,000 kw-hr (45% L.F.)

(*A*) New Steam Plant Installed	Annual Cost	(*B*) Bridgeport Hydro Project Developed	Annual Cost
1. Fixed charges and fixed operating cost on existing steam plants, tie lines, etc.	$ F	1. Fixed charges and fixed operating cost on existing steam plants, tie lines, etc.	$ F
2. Variable operating cost existing steam plants (4,340,000,000 − 1,025,000,000 = 3,315,000,000 kw-hr (@ 2.15 mills)	7 127 000	2. Variable cost existing steam plants (4,340,000,000 − 565,000,000) = 3,775,000,000 kw-hr @ 2.15 mills	8,116,000
3. New steam plant, Fig. 5, Chapter 12		3. New steam plant	None
(a) Fixed charges 180,000 kw @ $13.50 — $2,430,000			
(b) Fixed operating charges 180,000 kw @ $3.80 — 684 000			
(c) Variable operating charges 914,000,000 kw-hr @ 1.45 mills — 1 325 000			
Total annual cost new steam plant	4 439 000		
4. New hydro development	None	4. New hydro Bridgeport project Total annual cost (from Table 7, Chapter 13)	1,955,000
Total annual cost power supply with new steam plant in 1950	$11,566 000 plus F	Total annual cost power supply with Bridgeport hydro project in 1950	$10,071,000 plus F

Net annual saving in favor of installing Bridgeport hydro project = $11,566,000 − $10,071,000 = $1,495,000

References:

Section 11, Chapter 15.
Section 17, Chapter 15.
Fig. 5, Chapter 12
Section 10, Chapter 14.
Section 11, Chapter 14
Section 12, Chapter 14
Section 9, Chapter 13
Section 4, Chapter 13.
Section 14, Chapter 13
Fig. 2, Chapter 13.
Fig. 12, Chapter 14.
Table 7, Chapter 13.

The comparison in Table 1 has been somewhat idealized to provide a simple illustration, and costs that would be the same for both alternatives have not been entered in the table. The effect of the greater amount of energy that would be produced by the alternative steam plant is shown in Item 2 of the table. Usually the setups would be more complicated than that here shown. For instance, the investment for additional tie lines required under the two alternatives might be quite different. The principle, however, would be the same.

If, during the early years after installing the hydro only a part of the installed capacity were firm, it would be necessary to make setups for a number of years in the future and it might be found that installation of the Bridgeport hydro project would not be economically advisable until a later date when the load curve had grown and a larger installation would be firm capacity.

20. Peak-load Hydro Plants to Supersede Old Steam Plants. It is often convenient to consider peak-load hydro plants as competing for the service of the short-time and unusual peak loads of the system with the older steam plants that produce only a small amount of energy per year at a high cost. To make it economical to supersede such steam plants with peak-load hydro plants, the total additional annual cost of the hydro plant, including interest on the investment, must be less than the annual cost of the steam plant (excluding interest on the investment) that it is to supersede.

When a hydro plant is desired for peak-load purposes only, as, for instance, to produce 500 to 100 kw-hr per kw of installation per year, there are often many available sites where the unit capital cost per kilowatt is very low compared to the cost of most hydro plants (see Tables 1 and 2 of Chapter 13). For such a purpose we would locate a favorable site where the stream flow is very low compared to the proposed installation, where the pond is of ample size, and the cost of the dam insignificant compared to that of the power plant. Under favorable conditions such a plant can be constructed for \$60 to \$110 per kw.

In the present illustration, the total annual cost (including interest) on the hydro plant (capital cost, \$70 per kw) is assumed to be \$7.00 per kw of installed capacity. Assuming that energy production cost at the old steam plant that it is to supersede would be 4.5 mills per kw-hr and that the peak-load hydro plant would produce 1000 kw-hr per kw of capacity, the annual capacity cost of the peak load hydro would be $\$7 - (0.0045 \times 100) - \$2.50$ per kw. It is assumed that peak-load installations will not be made until such time as they would be firm capacity on the connected load curve. If now the "transmission liability" (see Section 6, Chapter 13) on the peak-load hydro plant is \$3.00 per kw per year, the total annual capacity cost of the peak-load hydro on a delivered basis in comparison with the old steam plant would be \$5.50 per kw. An examination of the data for old steam plants in Table 2 shows that all the plants listed have a fixed cost of operating in excess of \$7.00. When it is realized that, in addition, further savings will result from the discontinuance of old steam plants as payments to their replacement reserve (for repairs and renewals) will be stopped and also insurance and taxes on the old plant will be saved,* it is evident that there are situations where it would prove economical to supersede old steam plants with peak-load hydro plants capable of generating at least as much energy as the old steam plants produce.

21. Net Return on Investment in Pumped Storage Hydro Plants Superseding Old Steam Plants. The application of pumped storage hydroelectric plants (see Section 5, Chapter 11) superseding old steam plants for peak-load services is shown in Fig. 4, which gives a series of curves show-

* There would also be a credit due to scrap value and the sale of the site or its utilization for other purposes.

TABLE 2

Production Costs at Old Steam Plants That for the Most Part Supply Peak Loads and Serve as Reserve Capacity

(No Fixed Charges Are Included in Figures Given Below)

Plant	Approximate Date Built	Approximate Cost of Fuel per Million Btu *	Installed Capacity, kw	Fixed Cost of Operating per Kilowatt per Year	Variable Cost of Operating or Increment Cost of Energy, mills per kw-hr
1	1904	$0.1535	75,000	$ 8.60	$3.95
2	1905	0.1785	17,000	13.50	6.70
3	1910	0.1785	30,000	8.14	5.70
4	1910	0.1535	61,000	7.55	2.77
5	1911–1915	0.0954	65,000	11.46	4.10
6	1915	0.1785	40,000	10.05	4.38
7	1914 1917	0.1605	55,000	10.14	4.40
8	1916 1917	0 0954	33,000	11.40	4.01
9	1917–1921	0.1775	160,000	11.80	3.27
10	1917	0.1605	20,000	8.15	4.60
11	1919 1922	0 0630	25,000	8.80	2.30
12	1923	0.1605	40,000	8.60	3.80
13	1922 1925	0 1530	155,000	7.08	2.07
14	1923	0.0954	27,000	11.42	3.46
15	1926	0.1210	35,000	7.32	2.88
16	1926	0.1210	50,000	9.20	2.77
17	1924 1926	0.1005	22,000	8.22	1.60

* Bituminous coal of a good grade would have approximately 26,000,000 to 28,000,000 Btu per net ton; anthracite, 21,000,000 Btu per net ton; oil, 6,250,000 Btu per barrel. One barrel contains 42 U.S. gallons, with 7.7 to 8 lb per gal and about 18,500 Btu per lb.

ing the net percentage return on the capital cost of pumped storage hydroelectric plants for various capital costs of the hydro plants and various operating costs at old steam plants. For the old steam plants it is assumed that there is no return on the money invested therein. This is simply a special case of a peak-load hydro plant, as discussed in Section 20, used to supersede an old steam plant where the natural water supply for the hydro plant is insufficient or nonexistent and must therefore be supplemented or entirely provided by pumping.

Each of the old steam plants and the alternative pumped storage plant are assumed to operate on annual capacity factors of 15%. Any proposed pumped storage hydroelectric development requires a very thorough analysis of all the factors involved in order to determine whether or not it is eco-

nomically feasible, but Fig. 4 is interesting because it indicates to some extent the limits of the application of such plants for this purpose.

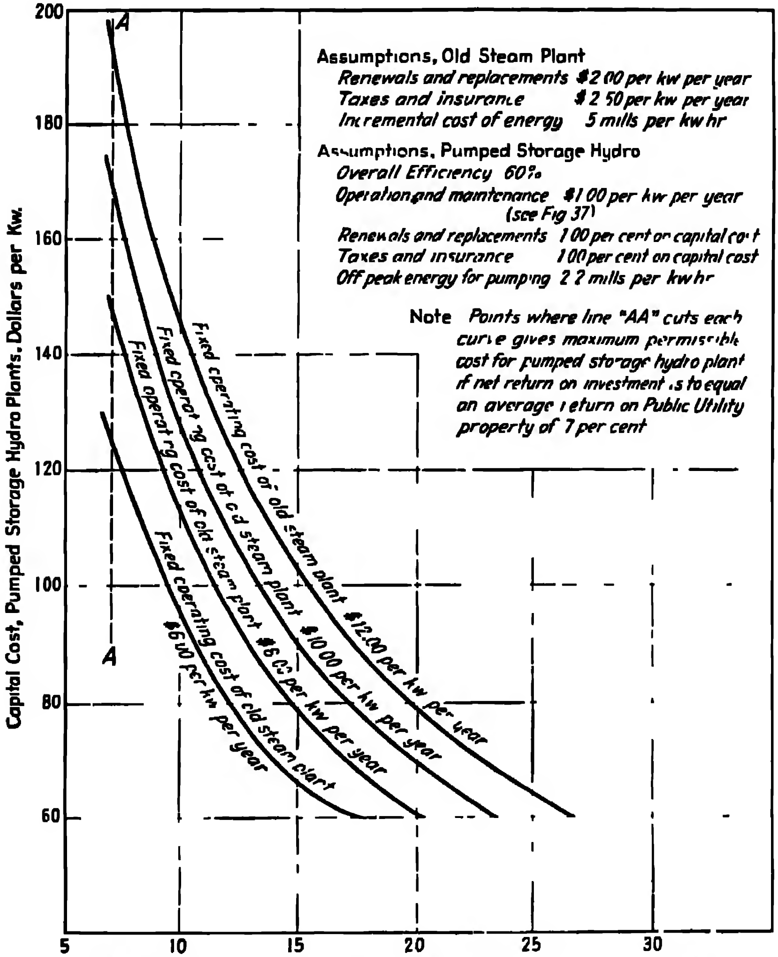

FIG. 4. Percentage return on investment in pumped storage hydro plants used to supersede old steam plants (annual capacity factor 15%).

22. Bibliography. Below is a list of books and articles dealing in whole or in part with the economic advisability of hydro installation.

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2. JOEL D. JUSTIN and WILLIAM G. MERVINE, *Power Supply Economics*, John Wiley & Sons, New York, 1934.

3. EZRA B. WHITMAN, "Combined Energy Generation," *Trans. A.S.C.E.*, Vol. 104, p. 1115, 1939.

4. W. F. UHL, "Cost of Energy 'Recapitulation,'" *Trans. A.S.C.E.*, Vol. 104, p. 1139, 1939.

5. "Cost of Energy Generation," *Trans. A.S.C.E.*, Vol. 104, 1939: discussion by JOEL D. JUSTIN, p. 1149; by H. G. GERDES, p. 1167; by LEVERETT S. LYON, p. 1173; by PHILIP SPORN, p. 1186; by L. N. McCLELLAND, p. 1188.

6. N. L. HARRINGTON and E. B. STROWGER, "Operating Methods and Problems of a Combined Hydro and Steam Electric System," *Trans. A.S.M.E.*, Vol. 61, No. 2, p. 89, February 1939; discussion by ALBION DAVIS, August 1939, p. 555.

7. JOEL D. JUSTIN, "Improvements in the Utilization of Energy," *Trans. A.S.C.E.*, Vol. 104, p. 971, 1939.

8. C. F. MERRIAM, "Improving Power System Economy by Combining Hydro with Steam," *Power*, September 1932, p. 127.

9. F. F. FOWLE, "Transmission of Remote Hydro Power More Costly than Coal Haul," *Electrical World*, June 20, 1936, p. 1918.

10. K. M. IRWIN and JOEL D. JUSTIN, "Economic Balance between Steam and Hydro Capacity," *Trans. A.S.M.E.*, Vol. 55, No. 2, p. 63, March 1933.

11. W. W. K. FREEMAN, "Pumped Storage Hydro-Electric Plants," *Trans. A.S.C.E.*, Vol. 94, p. 884, 1930; discussion by PAUL L. HESLOP, p. 910, and by A. H. MARKWART, p. 911.

CHAPTER 16

REPORTS

1. Introduction.* The report is the medium through which the engineer conveys his ideas to his client and his client's engineer. He has made his investigations, surveys, borings, studies, and preliminary design and as a result has reached some conclusion as to the most advisable course for his client to pursue under the given circumstances. He now attempts to state his conclusions with convincing clarity and supports them with a discussion of the data and considerations which led him to reach these conclusions. In general it is desirable that the report be accompanied by enough detailed data to substantiate the conclusions reached.

To a very large extent the ideas of the engineer find expression primarily in his reports, and executives and businessmen are likely to judge him thereby. If his reports are inconclusive and hedged about with many "ifs" and "ands," leaving engineering problems to be decided by the executives, the executives are likely to go elsewhere for such services next time. On the other hand, if his conclusions are tersely and concisely stated with convincing clarity and are borne out by the facts, he may obtain a life-long client. The ability to prepare a good report is one of the most important assets of the successful engineer.

This does not mean that reports should be prepared to please the client. The objective is to convince the client of the truth as the engineer has found it. Sometimes it may prove unpalatable and even productive of anger on the part of the client. Such a rare reaction may at times appear to be to the disadvantage of the engineer, but in the long run, entirely aside from the matter of principles, it never pays to color a report to please the recipient.

2. Purpose of Reports. Engineers are frequently called upon to prepare reports on water-power sites, projects, or partially or wholly completed developments or systems of developments, to show their value or advisability under certain conditions. Some reports deal only with some particular feature of the project, like the foundation conditions; the most advisable type of dam to use under the given conditions; the tailrace, the conduit, the most desirable type of units to adopt, etc. The usual purposes for which reports on hydro projects are required are as follows:

* It is assumed that the reader has studied Chapter 12, "Cost of Steam Power"; Chapter 13, "Cost of Hydro Power"; Chapter 14, "Market Requirements and Load Studies"; and Chapter 15, "Capacity of the Development and Economic Advisability of Hydro," before reading this chapter.

1. *Promotion.* To assist in the promotion of a project or to furnish engineering advice to a prospective investor in a project.

2. *Marketing a Site, Plant, or System.* To assist in the sale of a site, plant, or system or to furnish engineering advice to a prospective purchaser of same.

3. *Consolidations.* To estimate the value of the physical property of a system for purposes of stock allotment in proposed consolidations of several going concerns.

4. *Choice of Site.* To determine the best site to develop for certain market requirements.

5. *Determination of the advisability of construction* of a project or of any proposed improvement under existing economic conditions.

6. *Condemnation.* To estimate the value of a site, plant, or system in connection with condemnation proceedings.

7. *Physical Data.* To estimate the present or replacement value of the physical property of a plant or system.

3. Extent of the Report. The extent of the report varies in some measure according to the wishes of the client but, in general, with the extent that it is necessary for the engineer to go into details in order to substantiate the validity of his conclusions and recommendations. There is no merit in making the report voluminous; mere size impresses no one of experience. It is not usually advisable to include in a report all the detailed data utilized by the engineer in reaching his conclusions. The data can usually be summarized for the report. On the other hand, all utilized data should be available to the client.

Sometimes it is advisable to prepare a preliminary report for the perusal of the client, based on available data, a cursory examination of the site, and approximate estimates. Such reports, prepared in advance of adequate surveys and subsurface investigations, are necessarily highly tentative and must be hedged about with many reservations. Nevertheless, when made by an experienced engineer, they are frequently sufficient to show the probable feasibility or lack of feasibility of the project. If the preliminary report shows a value which is satisfactory to the client, it is superseded by a final report based on extended investigations and studies for a more accurate estimate of value.

4. Annual Depreciation. The annual charges on hydroelectric developments, including depreciation, are discussed in some detail in Chapter 13, and in Fig. 1 of Chapter 12 there are given curves for estimating annual payments to depreciation reserve with 5% and 7% money. Table 1 herein gives the same information, but for a range of interest rates from 4 to 9%. The table is derived from the following equation:

$$A = \frac{i}{(1 + i)^n - 1} \qquad [1]$$

where A = each annual payment, in dollars;

i = the interest rate, expressed as a decimal;

n = the period of years covering the useful life of the structure or piece of equipment.

If interest is paid semiannually, the annual payments will be only slightly less.

It has been contended that the proper rate to use is the average rate of return which the given industry makes annually on its investment. It is usual and recommended practice, however, under the sinking-fund method of figuring depreciation, to use a relatively low rate such as 4 or 5%. See Table 1. Actually, reserves are reinvested in the business, but theoretically, and sometimes actually, the depreciation reserve is a separate fund earning only a safe return.

TABLE 1

ANNUAL PAYMENTS REQUIRED TO ACCUMULATE ONE DOLLAR AT THE END OF A GIVEN NUMBER OF YEARS

Interest Compounded Annually

Number of Years	Interest Rate 3%	4%	5%	6%	7%	8%
2	0.493	0.497	0.488	0.485	0.482	0.480
3	0.324	0.320	0.317	0.314	0.311	0.309
4	0.239	0.235	0.232	0.228	0.225	0.222
5	0.189	0.184	0.181	0.177	0.174	0.171
6	0.153	0.151	0.147	0.143	0.140	0.136
7	0.130	0.126	0.123	0.119	0.115	0.112
8	0.111	0.109	0.105	0.101	0.098	0.094
10	0.087	0.0834	0.0795	0.0758	0.0724	0.0690
12	0.0705	0.0666	0.0628	0.0592	0.0559	0.0527
15	0.054	0.0499	0.0463	0.0429	0.0398	0.0368
20	0.037	0.0336	0.0303	0.0272	0.0244	0.0218
25	0.027	0.0240	0 0209	0.0182	0.0158	0.0137
30	0.021	0.0178	0.0150	0.0127	0.0106	0.00883
35	0.017	0.0136	0.0111	0.00900	0.00724	0.00580
40	0.013	0.0105	0.00828	0.00647	0.00501	0.00377
45	0.0108	0.00826	0.00627	0.00470	0.00350	0.00259
50	0.0080	0.00655	0.00478	0.00344	0.00246	0.00175

5. Accrued Depreciation. "Annual depreciation" of an item is defined as its annual loss of value. "Accrued depreciation" may be similarly defined as its accumulated loss in value at the end of a given period of years equal to or less than its estimated useful life. "Accrued depreciation reserve," as herein used, may be similarly defined as the accumulated replacement fund at the end of a given period of years.

The derivation of the amount of accrued depreciation reserve may be explained best by an example. Thus, from Table 1, it is evident that, in order to provide a fund of \$10,000 for the replacement of an item at the end of 20 years, it is necessary to set aside annually 2.44% of \$10,000, or \$244, at 7% compound interest. Suppose it is desired to know the accrued depreciation when the item is 10 years old. From Table 1 it is seen that \$0.0724 set aside annually at 7% compound interest will accumulate \$1 at the end of 10 years. Therefore, an annual deposit of \$244 will, at the end of 10 years, amount to $244 \times 1.00 \div 0.0724 = \$3370$, which is the accrued depreciation of the item at the end of 10 years.

The foregoing method of computing accrued depreciation is known as the "sinking-fund" method and is generally used by engineers in reports. It will be noted, in the previous example, that the item in question, having passed through one half of its estimated life, is only 33.7% depreciated. This is because the reserve accruals with compound interest will be more rapid in the latter part of the life of the item than in the first part.

The actual physical depreciation not being determinable, the sinking-fund method of estimating it has the most favorable features from many points of view. However, some commissions having jurisdiction over the accounting of public-service corporations have decreed that accrued depreciation, as affecting statements of capital account, shall be figured by what is known as the "straight-line" method. This method is based on the assumption that accrued depreciation is directly proportional to the age of the item. In other words, the accrued depreciation at a given time is equal to its first cost times the ratio of its age to its estimated life. Thus an item having passed through one half its estimated life is 50% depreciated.

Whereas engineers usually estimate accrued depreciation on the basis of the estimated cost of replacement, not on the first cost, accountants generally use the first cost only, and commissions, also, are based on first cost.

6. Legal Requirements. The engineer should obtain competent advice on federal and local legal requirements that affect the value of a plant or project. Such requirements are too numerous to describe here, and they differ materially according to location and the company's charter rights.

The Federal Power Commission has jurisdiction over all developments on navigable streams and those requiring the use of government lands. As "navigable" streams embrace not only those that are at present navigable but also those that can be made so by the construction of locks and dams, the jurisdiction of the Commission is theoretically unlimited and the practical limit has not been well defined. For such developments, permits must be requested and licenses obtained; all plans are subject to the approval of the Commission. Certain fees and rentals are demanded, and locks are sometimes required.

Several usual legal requirements are mentioned in Item VIII of Table 4. One state prohibits the exportation of power; another requires reservoirs and ponds to be cleared of trees and brush to the bottom instead of to low water; many states require dams to be built in accordance with certain specifications

which are quite radical; cemeteries may be condemned for flowage in some states and not in others; state lands, in many states, may not be used for a development under any circumstances, and in others, only by an act of legislature.

The right to interfere with the flow of the stream to the detriment of other users on the stream exists only under special circumstances. Therefore a project may not be allowed to operate at a low load factor if there are plants below that have no pond and are equipped to run only at a high load factor and therefore require a fairly continuous flow.

These and other requirements and limitations have sometimes been sufficient to prevent the development of otherwise excellent projects.

7. Rate of Return. Most reports on hydroelectric projects or features thereof deal with proposed expenditures. No one wishes to spend money unless it will produce a satisfactory return on the investment, and, therefore, reports should show in the terms preferred by the client what return may be expected from the proposed investment. Public utilities are limited almost everywhere by law or by the regulation of public bodies in the maximum return which they can make on their total investment.

In the United States this permissible maximum net return is seldom above 7%, and investors in public utilities would be happy indeed if the total investment really produced a net return of 7%. However, with any proposed expenditure, it is usually necessary that it should show a greater return than this because there is always the risk that things may not turn out as anticipated. Construction costs may be greater than estimated, and stream flow available may prove to be less. If a power company can develop a hydro project and have reason to believe that the investment will provide a net return of 10%, it would generally undertake the expenditure.

On the other hand, if the enterprise is speculative as, for instance, a project in undeveloped country where market conditions are uncertain and new industries must be attracted, the prospective return on the investment would have to be much greater to be attractive. (See also Section 10, Chapter 13.)

The sort of financial setup which should be used in the report varies greatly with the type of the report and with particular situations. Thus, in order to make it economically advisable to undertake a water-power development, a power company with a growing market may merely want to know that the hydro project will give them the required increment of power at a slightly lower annual cost than an alternative steam plant. On the other hand if the project is to be developed by a company that owns only that project and is planning on selling its output by contract to other companies, the promoters or owners will be interested directly in the net return on the investment which they can anticipate. More or less typical financial setups are given in Chapters 13 and 15.

8. Values for Future Development. If a project is not suitable for immediate needs or if its development must be postponed because there are

more favorable projects suitable for immediate market requirements, it may be purchased for use sometime in the near future. In this case the total amount invested in lands and water rights at the end of the period of idleness will be the sum of the following items:

(*a*) The amount paid for the lands and water rights.
(*b*) The interest on the purchase price during the period.
(*c*) Taxes on the value of the lands and water rights during the period (value usually assumed equal to purchase price).
(*d*) Interest on the money borrowed to pay the interest and taxes.

In other words, the interest and taxes will be compounded during the period.

Let n = the period of idleness in years;
i = the interest rate, expressed as a decimal;
t = the tax rate, expressed as a decimal;
P = the principal amount paid for lands and water rights;
C = the total investment at the end of the period.

Then, if interest is paid quarterly,

$$C = P\left(1 + \frac{i + t}{4}\right)^{4n} \qquad [2]$$

9. Estimates of Cost. The tendency is to estimate too low on all types of projects, and estimates of water powers are no exception. The greatest source of errors in estimating is the failure to provide sufficient time for adequate investigations and studies.

Table 2 contains a list of most of the important items which enter into the cost of a hydroelectric development. Such a list should be kept by engineers and consulted after each estimate to guard against omissions.

Manufacturers of turbines, generators, and other apparatus are always willing to give accurate estimates of cost, and such estimates are usually very reliable if sufficient time is taken for their preparation. Bids are frequently obtained from prospective contractors on the general construction items. Estimates based on these data, supplemented by the engineer's judgment, are the most acceptable to the prospective investor.

An estimate of cost is practically worthless unless based on first-hand knowledge of the estimator. An able engineer will not attempt to estimate the cost of an important item upon which he has had no experience without consulting someone who has. The use of published or offhand statements of unit costs on other jobs is to be emphatically discouraged. They seldom are reliable unless accompanied by an ample description of the costs included therein, as they sometimes show bare labor and material costs, excluding plant, miscellaneous, and other overhead expenses which are a large percentage of the total. Moreover, they frequently apply to conditions far different from those pertaining to the project. Published unit-price bids on other jobs are likely to be unbalanced.

TABLE 2

ESTIMATING FORM, HYDROELECTRIC POWER DEVELOPMENT

I. Preliminary expenses.
- (a) Previous expense and overhead.
- (b) Surveying and mapping.
- (c) Reports.
- (d) Borings and test pits.
- (e) Miscellaneous.
- (f) Office and camps (see also XXV).
- (g) Power-transmission line for construction purposes.
- (h) Temporary telephone.
- (i) Franchises.

II. Clearing site.
- (a) Removing structures and apparatus.
- (b) Removing trees and brush.
- (c) Grubbing.
- (d) Salvage.

III. Highways and bridges.
- (a) Relocation of existing.
- (b) For access to site.
- (c) For local construction purposes.
- (d) Local permanent roads and bridges.
- (e) Roads crossing conduits.
- (f) Special structures.

IV. Railroads and bridges.
- (a) Relocation of existing.
- (b) For access to site.
- (c) For local construction purposes (except as included in contractor's plant).

V. Damage to public utilities.

VI. Wharves.
- (a) Relocation of existing.
- (b) For access to site.
- (c) For local construction purposes.

VII. Ferries.
- (a) Relocation of existing.
- (b) For access to site.
- (c) For local construction purposes.

VIII. Taking care of water.
- (a) Earth excavation.
- (b) Rock excavation.
- (c) Embankment.

TABLE 2—*Continued*

ESTIMATING FORM, HYDROELECTRIC POWER DEVELOPMENT

(*d*) Cofferdam.
(*e*) Sheet piling.
(*f*) Round piles.
(*g*) Heel fill.
(*h*) Sand-bag fill.
(*i*) Pumping.
(*j*) Gates and sluices.
(*k*) By-pass conduit (see XII).
(*l*) Removal of cofferdam, etc.

IX. Reservoirs and pond.
(*a*) Preliminary (see I).
(*b*) Clearing site (see II).
(*c*) Highways and bridges (see III).
(*d*) Railroads and bridges (see IV).
(*e*) Wharves (see VI).
(*f*) Ferries (see VII).
(*g*) Drift barriers.
(*h*) Cemeteries.

X. Dams.
(*a*) Preliminary (see I).
(*b*) Clearing site (see II).
(*c*) Highways and bridges (see III).
(*d*) Railroads and bridges (see IV).
(*e*) Wharves (see VI).
(*f*) Ferries (see VII).
(*g*) Taking care of water (see VIII).
(*h*) Ice and trash fenders.
(*i*) Fishway.
(*j*) Log, ice, and trash chute.
(*k*) Sluice gates.
(1) Gates.
(2) Hoists.
(3) Sluice lining.
(4) Screens.
(5) Electrical equipment.
(*l*) Bridge on crest.
(*m*) Movable crest and operating mechanism.
(*n*) Air supply for (*m*) to prevent freezing.
(*o*) Lighting.
(*p*) Intake and equipment (see XI).
(*q*) Masonry dams,
(1) See X(*a*) to X(*q*), inclusive.
(2) Earth excavation (see also X(*q*) 12*a*)
(3) Rock excavation (see also X(*q*) 12*b*).
(4) Preparing foundations and grouting (see also X(*q*) 12*e*).

TABLE 2—*Continued*

ESTIMATING FORM, HYDROELECTRIC POWER DEVELOPMENT

(5) Bearing piles.
(6) Masonry (see also X(*q*) 12*d*).
(7) Backfill.
(8) Underdrainage.
(9) Steel reinforcement.
(10) Miscellaneous steelwork.
(11) Apron.
(12) Cutoff.
a. Earth excavation.
b. Rock excavation.
c. Sheet piling.
d. Masonry.
e. Grouting (see also X(*q*) 4).
(13) Water stops.
(14) Heel fill.
(15) Expansion joints.
(*r*) Timber dams.
(1) See X(*a*) to X(*q*), inclusive.
(2) Earth excavation.
(3) Rock excavation.
(4) Concrete footings.
(5) Bearing piles.
(6) Sheet piling.
(7) Grouting.
(8) Timbers.
(9) Sheeting.
(10) Stone filling.
(11) Earth filling.
(12) Miscellaneous steel, bolts, and pins.
(13) Heel fill.
(14) Apron.
(*s*) Earth dams.
(1) See X(*a*) to X(*p*), inclusive.
(2) Earth excavation (see also X(*s*) 7*a*)
(3) Rock excavation (see also X(*s*) 7*a*).
(4) Earth fill.
(5) Rock fill.
(6) Retaining walls.
(7) Core walls and cutoffs.
a. Earth excavation.
b. Rock excavation.
c. Masonry.
d. Steel reinforcement.
e. Water stops.
f. Puddle.
g. Sheet piling.
h. Grouting.

TABLE 2—*Continued*

ESTIMATING FORM, HYDROELECTRIC POWER DEVELOPMENT

(8) Slope protection.
(9) Sodding and seeding.
(10) Gutters and drains.
(11) Roadway surfacing.
(12) Outlet tower (see XI).
(13) Bridge to tower (see XI).
(14) Outlet under embankment (see XII).
(15) Stripping and grubbing.
(16) Drains.
(17) Gravel filters.
(18) Upstream blanket.
(19) Gunite.
(20) Grout cap.

(*l*) Isolated spillways.
(1) See previous applicable items.
(2) Reinforced-concrete paving.
(3) Stilling basins.
(4) Walls and flumes.
(5) Riprap.
(6) Getaway canal.

XI. Intakes.

(*a*) Preliminary (see I).
(*b*) Clearing site (see II).
(*c*) Highways and bridges (see III).
(*d*) Railroads and bridges (see IV).
(*e*) Wharves (see VI).
(*f*) Ferries (see VII).
(*g*) Taking care of water (see VIII).
(*h*) Ice and trash fender and chutes.
(*i*) Bridge on top.
(*j*) Lighting, wiring, and ducts.
(*k*) Structure.
(1) Earth excavation (see XI(*k*) 10*a*).
(2) Rock excavation (see XI(*k*) 10*b*).
(3) Preparing foundations and grouting (see also XI(*k*) 10*c*).
(4) Bearing piles.
(5) Masonry (see also XI(*k*) 10*d* and XI(*k*) 15).
(6) Backfill.
(7) Underdrainage.
(8) Steel reinforcement.
(9) Miscellaneous steelwork
(10) Cutoff.
a. Earth excavation.
b. Rock excavation.
c. Sheet piling.
d. Masonry.

TABLE 2—*Continued*

ESTIMATING FORM, HYDROELECTRIC POWER DEVELOPMENT

c. Grouting (see also XI(*k*) 3).

(11) Water stops.

(12) Air inlets.

(13) Superstructure (see XIV).

a. Housing over air inlet.

b. Housing over gates.

(14) Heating.

(15) Retaining walls (see also XI(*k*) 5).

(*l*) Equipment.

(1) Racks and supports.

(2) Main gates and supports.

(3) Main gate hoists.

(4) Exciter gates and supports.

(5) Exciter gate hoists.

(6) Filler gates and supports.

(7) Filler gate hoists.

(8) Electrical operation with cables and ducts

(9) Miscellaneous steel.

(10) Stoplogs and supports.

(11) Miscellaneous equipment.

(12) Mechanical rakes.

(13) Cranes.

(14) Air supply to prevent freezing.

XII. Conduits.

(*a*) Preliminary (see I).

(*b*) Clearing site (see II).

(*c*) Highways and bridges (see III).

(*d*) Railroads and bridges (see IV).

(*e*) Wharves (see VI).

(*f*) Ferries (see VII).

(*g*) Taking care of water (see VIII)

(*h*) Intakes (see XI).

(*i*) Culverts.

(*j*) Sewers.

(*k*) Cattle crossings.

(*l*) Settling basins.

(*m*) Waste weirs (see X).

(*n*) Waste channels.

(*o*) Bridges and trestles for conduits.

(*p*) Fencing.

(*q*) Canals.

(1) See XII(*a*) to XII(*p*), inclusive.

(2) Earth excavation.

(3) Rock excavation.

(4) Embankment (see X(*s*)).

(5) Lining and slope protection.

TABLE 2—*Continued*

ESTIMATING FORM, HYDROELECTRIC POWER DEVELOPMENT

- (6) Cable crossings.
- (7) Drops.
- (8) Reinforcing steel.
- (9) Retaining walls.
- (10) Wall drains.
- (11) Backfill.
- (12) Outside walls (see X).
- (13) Aqueducts (see XII(*t*)).
- (14) Emergency gates.

(*r*) Pipelines.
- (1) See XII(*a*) to XII(*p*), inclusive.
- (2) Earth excavation for bench; for piers
- (3) Rock excavation.
- (4) Embankment (see X(*s*)).
- (5) Covering and refill.
- (6) Pipe materials.
- (7) Circumferential stiffeners.
- (8) Cradles.
- (9) Concrete covering under embankments
- (10) Piers and sills.
- (11) Anchorages.
- (12) Reinforcing steel.
- (13) Painting and creosoting.
- (14) Expansion joints.
- (15) Air inlets.
- (16) Air outlets.
- (17) Blowoffs.
- (18) Drainage of pipe.
- (19) Underdrainage.
- (20) Valves.
- (21) Valve houses.
- (22) Relief valves.
- (23) Manholes.
- (24) Special connections.
- (25) Surge tank (see XII(*v*)).

(*s*) Penstocks (see XII(*r*)).

(*t*) Flumes.
- (1) See XII(*a*) to XII(*p*), inclusive.
- (2) Earth excavation.
- (3) Rock excavation.
- (4) Embankment (see X (*s*)).
- (5) Flume materials.
- (6) Flume footings.
- (7) Foundation drainage.
- (8) Drops.
- (9) Reinforcing steel.
- (10) Painting.

TABLE 2—*Continued*

ESTIMATING FORM, HYDROELECTRIC POWER DEVELOPMENT

(11) Expansion joints.
(12) Emergency gates.
(*u*) Tunnels.
(1) See XII(*a*) to XII(*p*), inclusive.
(2) Portals.
a. Earth excavation.
b. Rock excavation.
c. Refill.
d. Bank protection.
e. Paving.
f. Masonry.
g. Steel reinforcement.
h. Drains.
(3) Tunnel earth excavation
(4) Tunnel rock excavation.
(5) Bracing.
(6) Masonry lining.
(7) Steel lining.
(8) Crown fill.
(9) Invert fill.
(10) Cutoffs around lining.
(11) Lining drains.
(12) Invert drains.
(13) Grouting.
(14) Reinforcing steel.
(15) Air inlets.
(16) Air outlets.
(17) Blowoffs.
(18) Special connections.
(19) Valves.
(*v*) Surge tanks.
(1) See XII(*a*) to XII(*p*), inclusive.
(2) Earth excavation.
(3) Rock excavation.
(4) Refill and grading.
(5) Foundation drains.
(6) Tank drains.
(7) Concrete footings.
(8) Reinforcing steel.
(9) Surge tank and tower.
(10) Frostproofing.
(11) Painting.
(12) Heating apparatus.

XIII. Powerhouse substructure.
(*a*) Preliminary (see I).
(*b*) Clearing site (see II).

TABLE 2—*Continued*

ESTIMATING FORM, HYDROELECTRIC POWER DEVELOPMENT

(*c*) Highways and bridges (see III).
(*d*) Railroads and bridges (see IV).
(*e*) Wharves (see VI).
(*f*) Ferries (see VII).
(*g*) Taking care of water (see VIII).
(*h*) Earth excavation.
(*i*) Rock excavation.
(*j*) Preparing foundations and grouting.
(*k*) Bearing piles and shoes.
(*l*) Underdrainage system.
(*m*) Cutoff (see X(*q*) 12), steel sheet piling.
(*n*) Refill and embankment.
(*o*) Masonry.
(*p*) Steel reinforcement.
(*q*) Miscellaneous steelwork.
(*r*) Powerhouse drains and valves
(*s*) Intake equipment (see XI(*l*)).
(*t*) Air inlets.
(*u*) Ice and trash fenders.
(*v*) Water stops.
(*w*) Water supply.
(*x*) Draft-tube forms.
(*y*) Drainpiping.
(*z*) Stoplogs.
(*aa*) Stoplog supports.
(*bb*) Floor hardener.

XIV. Powerhouse superstructure.
(*a*) Structural steel.
(*b*) Miscellaneous steel.
(*c*) Walls and partitions.
(*d*) Floors.
(*e*) Floor surfacing and hardeners.
(*f*) Floor covering.
(*g*) Sills and lintels.
(*h*) Roof.
(*i*) Roof covering.
(*j*) Gutters, leaders, and flashing.
(*k*) Steel reinforcement.
(*l*) Doors, windows, and skylights.
(*m*) Hardware.
(*n*) Painting and finishing.
(*o*) Ventilators.
(*p*) Plumbing and water.
(*q*) Lighting.
(*r*) Heating.
(*s*) Bus structure (if not in XVII).

TABLE 2—*Continued*

ESTIMATING FORM, HYDROELECTRIC POWER DEVELOPMENT

(*t*) Switch cells.
(*u*) Fire protection.
(*v*) Office equipment.
(*w*) Cornices and supports.

XV. Hydraulic equipment.
(*a*) Main turbines.
(*b*) Exciter turbines.
(*c*) Main governors, pumps, tanks, and piping.
(*d*) Exciter governors, pumps, tanks, and piping.
(*e*) Governor oil.
(*f*) Switchboard control.
(*g*) Hand control.
(*h*) Relief valves (see XII(*r*) 20).
(*i*) Water-cooling system for bearings.
(*j*) Oiling system.

XVI. Electrical equipment.
(*a*) Main generators.
(*b*) Exciter generators.
(*c*) Low-tension switches.
(*d*) Low-tension bus-bars.
(*e*) Switchboard.
(*f*) Wiring and ducts.
(*g*) Transformers.
(*h*) High-tension switches.
(*i*) High-tension bus-bars.
(*j*) Lightning arresters.
(*k*) Transformer oil.
(*l*) Oil filters and pumps.
(*m*) Oiling system.
(*n*) Oil cooling system.
(*o*) Motor-generator sets.
(*p*) Storage batteries.

XVII. Miscellaneous equipment.
(*a*) Crane and motor.
(*b*) Pumps and motors.
(*c*) Eductors.
(*d*) Machine shop.
(*e*) Transformer truck and track.
(*f*) Air compressor.
(*g*) Fire pumps.
(*h*) Office equipment.
(*i*) Oil treatment.
(*j*) Miscellaneous oil, air, and water piping.
(*k*) Air conditioning.

TABLE 2—*Continued*

ESTIMATING FORM, HYDROELECTRIC POWER DEVELOPMENT

(*l*) Heat exchangers.
(*m*) Draft-tube gates and hoists.
(*n*) Draft-tube pumps.
(*o*) House water supply.
(*p*) Sanitation.

XVIII. Testing and starting.

XIX. Tailrace.

(*a*) Preliminary (see I).
(*b*) Clearing site (see II).
(*c*) Highways and bridges (see III)
(*d*) Railroads and bridges (see IV)
(*e*) Wharves (see VI).
(*f*) Ferries (see VII).
(*g*) Taking care of water (see VIII).
(*h*) Earth excavation.
(*i*) Rock excavation.
(*j*) Embankment and refill.
(*k*) Paving.
(*l*) Sheet piling.
(*m*) Slope protection.
(*n*) Retaining walls.
(*o*) River walls and crib.

XX. Outdoor Substation Structure:

(*a*) Preliminary (see I).
(*b*) Clearing site (see II).
(*c*) Highways and bridges (see III).
(*d*) Railroads and bridges (see IV).
(*e*) Wharves (see VI)
(*f*) Ferries (see VII).
(*g*) Taking care of water (see VIII).
(*h*) Earth excavation.
(*i*) Rock excavation.
(*j*) Embankment and refill.
(*k*) Slope protection.
(*l*) Retaining walls.
(*m*) Concrete footings.
(*n*) Concrete floor paving.
(*o*) Superstructure (see also XIV).
(*p*) Fence.
(*q*) Transformer track (see also XVII(*e*)).
(*r*) Drains.
(*s*) Structural steel.
(*t*) Equipment (see XVI).

TABLE 2—*Continued*

ESTIMATING FORM, HYDROELECTRIC POWER DEVELOPMENT

XXI. Indicating and recording devices for hydraulic purposes.
- (*a*) Venturi meters.
- (*b*) Pitot tubes.
- (*c*) Measuring weirs (see X).
- (*d*) Water-stage indicators.
- (*e*) Water-stage alarms.
- (*f*) Miscellaneous metering devices.

XXII. Permanent transmission line and telephone line.
- (*a*) Foundations.
- (*b*) Anchors.
- (*c*) Steel towers.
- (*d*) Poles.
- (*e*) Cross-arms.
- (*f*) Insulators.
- (*g*) Conductors.
- (*h*) Ground wire and grounds.
- (*i*) Stay guys and anchors.
- (*j*) Telephone line and insulators.
- (*k*) Transformers and meters.
- (*l*) Special crossings.
- (*m*) Sectionalizing stations.

XXIII. Temporary transmission line and telephone line.

XXIV. Substations (see XVI and XX).

XXV. Permanent quarters.
- (*a*) Operators' houses.
- (*b*) Guest house.
- (*c*) Garage and stables.
- (*d*) Water supply and sanitation.
- (*e*) Outside lighting.

XXVI. Construction overhead (unless included in unit costs).
- (*a*) Freight, haulage, erection, and removal of construction plant.
- (*b*) Organization and overhead expenses.
- (*c*) Storeroom salaries and expenses.
- (*d*) Watching, lighting, and guarding.
- (*e*) Provisions for safety to persons and property.
- (*f*) Accidents and damage.
- (*g*) Plant investment less salvage, or plant rental, or plant depreciation.
- (*h*) Plant operation and maintenance and repairs.
- (*i*) Small tools.
- (*j*) Garage, commissary, camp, and other auxiliary feature investment and operation.

TABLE 2—*Continued*

ESTIMATING FORM, HYDROELECTRIC POWER DEVELOPMENT

(*k*) Fire, payroll, and personal injury insurance.
(*l*) Contractor's bond.

XXVII. Local general charges.
(*a*) Engineering and office equipment.
(*b*) Engineering salaries and expenses.
(*c*) Engineering supplies.
(*d*) Office salaries and expenses.
(*e*) Fire, payroll, and personal injury insurance
(*f*) Engineers' quarters.
(*g*) Auxiliary features (see XXVI(*j*)).

XXVIII. Engineering and miscellaneous.
(*a*) Home office engineering, supervision, purchasing, inspection, and overhead.
(*b*) Consulting engineering.
(*c*) Legal expenses.
(*d*) Organization expenses.
(*e*) Client's office salaries and expenses.
(*f*) Entertaining.
(*g*) Royalties, franchises, and licenses.

XXIX. Allowance for unforeseen contingencies and omissions.

XXX. Real estate, rights of way, flowage rights, water rights, and cemeteries.

XXXI. Interest during construction.

XXXII. Taxes during construction.

XXXIII. Interest and taxes on cost of real estate, rights of way, flowage rights, and water rights during period of idleness prior to construction.

XXXIV. Working capital.

An allowance for unforeseen contingencies, omissions, miscellaneous items not large enough to be estimated separately, and possible errors in both estimated costs and quantities is usually provided at or near the end of the estimate. The amount depends upon the extent of the data available and the probable accuracy of the investigations and studies. It varies between 10 and 20% of the total estimated cost.

Interest during construction is approximately the interest on the total estimated cost for about one half the construction period; but, for projects taking several years to build, it is advisable to estimate a progress schedule and figure the interest charges more accurately.

Working capital is included only if the returns from power sales will not be quick enough to provide money for running expenses when the plant first starts up. Its inclusion is probably necessary only for a new company.

10. Arrangement and Wording of Report. Every report should be accompanied by a letter of transmittal which merely states the authority for the report and the number of copies that have been forwarded and to whom. The report should be introduced by a statement of its purpose and scope. Usually, in engineering reports, this is followed by a brief summary of conclusions and recommendations, after which are given, in as much detail as required, a logical discussion of the various factors involved and a description of the investigations and studies of the engineer. Maps, drawings, diagrams, lengthy tabulations, and similar exhibits are commonly placed in an appendix.

A table of contents giving a list of sections and the page numbers on which they commence should be included, together with a list of figures and tables. If the report is lengthy an alphabetical index of subjects should be added. Margins should be wide, and binding should be at the left-hand edge so that the report will open like a book. A common fault is to have the left-hand margin so narrow, especially on pages with figures and tables, that material at the extreme left is not visible after binding. This difficulty can be obviated by making sure that all left-hand margins are at least 1½ in. wide.

Figures and drawings accompanying and forming a part of the report should be assembled at the end of the typed material and should be bound with the report. If possible they should be limited to a width of 11 in. When wider sheets than this are unavoidable, the folds should be so placed that the reader does not have to do any unfolding to read the title and number of the figure or drawing.

Every report should carry the date on which it is submitted and the signature of the engineer responsible for it. This would seem axiomatic, but the authors have seen many reports from which one or the other was missing. It should never be necessary to read more than the first paragraph of a report to learn what it is about or more than the first few pages to find out what the engineer's conclusions and recommendations are. The busy executive to whom the report is addressed frequently reads only the "Purpose of the Report" and the "Summary of Conclusions and Recommendations," then thumbs it over, takes a look at the figures, makes a few notes, and turns the report over to his engineer for study and for favorable or unfavorable recommendation.

Neatness is an essential feature of a good report, as a poorly typed and bound report invariably carries with it an impression of inaccuracy. The typing should be double spaced on $8\frac{1}{2} \times 11$ in. sheets of good-quality paper and should be legible. The various subjects treated should be numbered and underscored as side headings, and subheads should be added where helpful.

The engineer should not use in his report technical terms that are not known to those for whom the report is intended unless such terms are fully explained; it is frequently advisable to include a glossary of technical terms

with definitions. Reports to laymen should differ materially in this respect from those to an engineer.

11. Subject Matter of Reports. A report is essentially an opinion. However, the client has a right to expect that, unless otherwise specifically noted, all statements in the report are based on adequate and careful investigations by the engineer.

It is not always possible for the engineer to base his report entirely on information that has been verified by himself or his assistants. Therefore, when necessary assumptions must be made, they should be clearly stated to be such, and all unverified data should be carefully scrutinized to determine their probable accuracy. The report should contain a statement of the authority or the source of the unverified information and, if advisable, recommendations to the effect that verification of such information is necessary before the report can be accepted in its entirety.

The engineer should be careful not to pass upon the validity of contracts, franchises, and similar documents. The client's attention should be called to all assumptions that are of a legal nature or for any other reason are outside the province of the engineer, so that they can be verified by the proper authorities.

The usual qualifications in the report of unverified data are similar to the following: "I am informed by your superintendent that . . ." or "According to the publications of the Weather Bureau . . ."

The engineer should also be careful to indicate the probable accuracy of his conclusions as affected by the data available. It is impossible to make exact estimates of cost of development, as they are affected by probable labor rates, probable floods, the amount of good weather during construction, and other influencing conditions which are impossible to predict. Consequently the engineer is compelled to base his conclusions to some extent on pure judgment. The experienced engineer will therefore form his own conclusions as to the values derived from poor, average, and good working conditions, even if only one value is given in his report. He will, however, always state in his report conditions on which the value is based.

Many reports are improperly qualified by the statement that "under normal and proper conditions, the conclusions will be substantiated." Other reports frequently state that "the estimates and conclusions are conservative," without giving the degree of conservatism. A large corporation, promoting a number of projects, may be content with statements of *probable* value based on *average* conditions, on the assumption that, if the conditions are worse on one project they will be better on another, and thus prove satisfactory as a whole.

On the other hand, a client investing in only one project will not be satisfied with a value based on average conditions. With bad luck, the project may fall short of being a successful enterprise. In such cases the conclusions in the report should be based on conditions worse than the average. Often

it may be advisable to give the client values based on two possible extremes as well as a probable value; this will allow him to take whatever gamble he desires, if the worst value is not entirely satisfactory.

For cases where, owing to differences in runoff, the output of the development will be greater in some years than in others, the usual basis of the report is the result for the average year. It is advisable, however, to give also the result for a minimum year if the result for several lean years immediately after starting the plant might cause embarrassment to the company.

There is an inherent desire on the part of promoters to have the engineer base his report on as favorable assumptions as the available data will permit. On the other hand, the attitude of investors is one of conservatism. The ethics of the engineering profession requires that the engineer's opinion, as expressed in the report, should be unbiased and should not depend upon whether the report is for a promoter or an investor. This is not a statement that the engineer's opinion should be an honest one; it is merely another way of expressing the need for proper qualification or explanation of such opinion as described above.

No engineer is an expert in all branches of his profession. It is never an admission of incompetency for an engineer to request consultations with other engineers having experience along special lines. Reports of geologists, contractors, foresters, foundation experts, and others may properly be appended to the engineer's main report.

12. Outline of the Report. After the engineer has made surveys and subsurface investigations of the sites involved (see Chapter 7), he is prepared to make his studies of how best to utilize the site, his estimates of cost under various alternatives, and his study of market conditions, etc. (See Chapters 12 to 15, inclusive.) From these he is able to determine the most advisable course for his client to pursue.

He is then ready to prepare his report. At this stage it is well to make an outline of the report showing the subjects to be discussed and their logical arrangement in the report. It usually consists of a list of the side headings or sections which it is intended to use in the report. This outline is generally altered from time to time as judgment dictates during the preparation of the report.

The following are typical headings for sections in a complete report on a project. These can be further subdivided as shown in Table 4.

1. Purpose and scope of report.
2. Summary of conclusions.
3. Corporate history of existing company.
4. Charters, franchises, and other assets.
5. Physical features of the watershed.
6. Water supply.
7. Head available.
8. Power available.
9. Power market.

10. Description of the site and existing property.
11. Proposed new work and extensions.
12. Description of field investigations.
13. Estimate of money required.
14. Estimate of gross annual income.
15. Estimate of annual charges.
16. Financial statement showing the value of the project.

Items 13, 14, and 15 are used in compiling the financial statement of Item 16.

The following are the section headings of another report prepared for the executives of a power company which was considering the advisability of purchasing certain water-power sites on a river held by a promoter, on which the promoter had already done considerable engineering work.

1. Purpose and scope of the report.
2. Summary of conclusions and recommendations.
3. List of important data from this report.
4. Outline of promoter's scheme of development.
5. Water supply and power available.
6. Promoter's cost estimate.
7. Development of some head previously investigated.
8. Riparian rights.
9. Necessity of acquiring rights of owners by-passed by diversion.
10. Power projects below diversion.
11. Utilization of blank river hydro power.
12. Plant capacities.
13. Estimated loads and suggested additional capacities.
14. Maximum demand week load curve.
15. Alternative hydro projects.
16. Transmission required.
17. Cost of delivered blank hydro power with installation of 130,000 kw.
18. Cost of delivered blank hydro power with installation of 200,000 kw.
19. Cost of alternative steam power.
20. Total annual cost of power supply with blank river hydro power added.
21. Total annual cost of power supply.
22. Increase in net income due to adding blank river hydro.

13. Records and Check Lists for Reports. All computations for reports should be fully explained and indexed. Frequently, the same project comes up at a later date, and it is sometimes quite difficult to find out how one arrived at all the conclusions of a report several years old. The authors have found that it is convenient to use summary sheets similar to those shown in Table 3 so as to have computed or determined data relative to the project or projects discussed in the report available in convenient form during and subsequent to the preparation of the report.

Table 4 has been found useful both as a guide in the preparation of the outline of the report and as check list after the preliminary draft of the report has been completed in order to see that everything pertinent has been covered.

TABLE 3

A FORM FOR THE COMPILATION OF PROJECT DATA

No.	Item		Unit	Reference
1	Name			
2	File Number of Calculations			
3	Date of Calculations			
4	River			
5	Location			
6	Owner	7 Promoter		
8	Purpose of Calculations and Accuracy			
9	Description of Development and Remarks			
10	Runoff Data			
11	Drainage Area		sq mi	
12	Per Cent of Flow Owned by (6)		%	
13	Assumed Storage	Billion cu ft Regulated for		
14	Location of Storage and Reference to Report			
15	Ownership of Storage			
16	Reference to Duration Curve of Runoff			
17	Tabulation of Runoff as Shown by Duration Curve			

No.	1 DESIGNATION	2 PER CENT OF TIME	3 SEC-FT FLOW	4 SEC-FT YEARS
18	Primary	100%		
19	Secondary	[illegible]%		
20	Pri plus Sec	[illegible]%		
21	Dump	%		
22	Total			

No.	Item			Unit	Reference
23	Floods				
24	50-year flood	sec-ft	W S at Dam El		
25	100-year flood	sec-ft	W S at Dam El		
26	1000-year flood	sec-ft	W S at Dam El		
27	Pondage Data				
28	Area of Pond at Permanent Crest of Dam (47)			acres	
29	Usual Maximum Drawdown of Pond below (47 or 48)			ft	
30	Capacity of Pond at (29)			acre ft	
31	Capacity of Pond at (29) in Flow days				
32	Primary Flow days				
33	Primary plus Secondary Flow days				
34	Mean Drawdown Corresponding to (29) below (47 or 48)			ft	
35	Possible Maximum Drawdown of Pond below (47 or 48)			ft	
36	Capacity of Pond at (35)			acre ft	
37	Effect on Pondage of the Operation of Upper Plants				
38	Load Factor and Load Curve				
39	Reference to Pondage Limitations				
40	Adopted Load Factor for Primary plus Secondary Power				
41	At Shaft			%	
42	At Switchboard			%	
43	For Delivered Power			%	
44	Elevations				
45	Datum of Elevations				
46	Conversion Factor to get U S G S Datum			ft	
47	Permanent Crest of Dam		El		
48	Top of Temporary Flashboards		El		
49	Top of Concrete Non-overflow Dam		El		
50	Top of Embankment at Dam		El		
51	Average Tailwater at Next Plant above		El		
52	Average Headwater Surface		El		
53	Average Tailwater Surface		El		
54	High Tailwater Surface for	Year Flood (23)	El		
55	Minimum Tailwater Surface		El		
56	Head				
57	Nominal Productive Head (Exact Information Lacking)			ft	
58	Gross Head with Full Pond			ft	
59	Average Gross Head			ft	
60	Productive Head			ft	
61	Minimum Net Head at Peak Demand Capacity (73)			ft	
62	Maximum Net Head at Full Load of One Unit (76)			ft	
63	Maximum Net Head at Full Load of Plant (77)			ft	

TABLE 3—*Continued*

A Form for the Compilation of Project Data

No.	Item	Col 1 Average Load	Col 2 Peak Load	Reference
64	Installation			
65	Assumed Turbine Efficiency	%	%	
66	Assumed Generator Efficiency	%	%	
67	Assumed Transformer Efficiency	%	%	
68	Assumed Transmission Efficiency	%	%	
69	Assumed Transformer Efficiency	%	%	
70	Assumed Other Equipment Efficiency	%	%	
71	Assumed Total Efficiency	%	%	
72	Average Output of Primary plus Secondary from (60) and Col 3 of (20)		shaft hp	
73	Peak Demand Capacity of Primary plus Secondary from (72) and (40) The Turbines Must Have This Capacity at (61)		shaft hp	
74				
75				
76	Maximum Capacity of Each of Units at (62) (Maximum Load into Generator)		shaft hp	
77	Maximum Capacity of Plant at (63)		shaft hp	
78	Maximum Discharge of Plant at (63)		sec-ft	
79	Kw Rated Capacity of Each of Generators		kw	
80	Kva Rated Capacity of Each of Generators at % Power Factor		kva	

No.	Item	Switchboard	Delivered
81	Output in kw hr per annum		
82	Primary		
83	Secondary		
84	Total Primary plus Secondary		
85	Dump		
86	Total		

No.	Item		Alternatives 1	2	3
87	Cost of Development				
88	Lands and Water Rights for Sale		$	$	$
89	Additional Lands and Water Rights Required		$	$	$
90	Construction Cost Exclusive of Transmission		$	$	$
91	Cost of Transmission to ___		$	$	$
92	Other Cost		$	$	$
93	Total Cost of Development		$	$	$
94	Total Cost per hp of Peak Demand (73)		$	$	$
95	Annual Operating Charges		$	$	$
96	Operators				
97	Powerhouse	$			
98	Outside Men	$			
99	Transmission	$			
100	Substations	$			
101	Miscellaneous	$		$	$
102	Maintenance and Repairs				
103	Power Development	$			
104	Transmission	$			
105	Substations	$			
106	Miscellaneous	$	$	$	$
107	Taxes		$	$	$
108	Insurance		$	$	$
109	Depreciation		$	$	$
110	Storage Charge		$	$	$
111	Management		$	$	$
112	Miscellaneous Charges		$	$	$
113	Credit Miscellaneous Income		$	$	$
114	Total Annual Charges		$	$	$
115	Interest Charges at % of (93)		$	$	$
116	Total Cost of Producing Power (114) + (115)		$	$	$
117	Cost of Producing Power, mills per kw hr from (116) and (81)				
118	Where Energy is Measured				
119	Primary Output (82)				
120	Secondary Output (83)				
121	Total Primary plus Secondary (84)				
122	Dump Output (85)				
123	Total Primary, Secondary and Dump (86)				

124 General Notes

TABLE 4

Combined Outline of Essential Data Required for a Report on a Plant or a Project

I. Scope of report.
   1. Client.
   2. Name and general location of plant or project
   3. Purpose of report.
   4. Type of report desired.
   5. Previous reports and studies.

II. Corporate history of existing company
   1. General information.
   2. Officers.
   3. Directors.
   4. Controlling interests.

III. Charters, franchises, and other assets.
   1. Charters.
      (a) Date, life, and powers.
      (b) Amendments.
   2. Franchises.
      (a) Date and life.
      (b) Rights secured and pending.
      (c) Conditions and restrictions.
      (d) Competitive franchises, with particulars.
   3. Other incorporeal assets.
      (a) Contracts.
         (1) Date and life.
         (2) Between what parties.
         (3) Conditions.
      (b) Leases and mortgages.
         (1) Date and life.
         (2) Between what parties
         (3) Conditions.
   4. Charters and franchises required for proposed new work and extensions.

IV. Physical features.
   1. Location and name of river.
   2. Nearest railroad point.
   3. Transportation facilities to site, existing and proposed.
   4. Drainage area.
   5. Topographical conditions of the watershed.
   6. Geological conditions of the watershed.
   7. Climatic conditions.
      (a) Range in temperature.
      (b) Variations in precipitation by seasons.
   8. Foundations (see also XII 5(e)).
   9. Existing storage, controlled and not controlled.
   10. Water supply.

TABLE 4—*Continued*

COMBINED OUTLINE OF ESSENTIAL DATA REQUIRED FOR A REPORT ON A PLANT OR A PROJECT

(*a*) Available stream gagings.
(*b*) Precipitation records.
(*c*) Average precipitation.
(*d*) Evaporation data
(*e*) Estimated present stream flow.
(*f*) Estimated stream flow after proposed storage.
(*g*) Estimated flood characteristics.
(*h*) Effect of plants above on variations in stream flow.
(*i*) Requirements of plants below, restricting variations in stream flow.
(*j*) Floating substances in the water.
  (1) Ice, surface, frazil, and anchor.
  (2) Effect of proposed new ponds on ice.
  (3) Silt.
  (4) Débris
  (5) Acids and other impurities.
(*k*) Condition of dams above if failure will cause damage.

V. Elevations.
1. Datum of elevations
2. Conversion factor to get U. S. G. S. datum.
3. Permanent crest of spillway, existing or proposed.
4. Top of temporary flashboards.
5. Top of concrete non-overflow dam.
6. Top of dam embankment.
7. Average tailwater of next plant above, if affected.
8. Average tailwater surface
9. Highest tailwater surface.
10. Lowest tailwater surface.

VI. Head.
1. Gross head.
2. Productive head (see definition, Chapter 9, Section 1).
3. Minimum net head.
4. Maximum net head.
5. Rating curve of tailrace.
6. Effect of possible ice gorges on tailrace elevation.

VII. Maps, profiles, photos, drawings, and literature.
1. Maps and profiles.
  (*a*) General map of surrounding territory covering sites, neighboring cities, railroads, highways, elevation above sea level, etc., preferably a contour map.
  (*b*) Map showing nature and size of drainage basin, giving elevations above sea level, preferably a contour map.
  (*c*) Map of reservoir sites showing contours at various stages of water surface.

TABLE 4—*Continued*

COMBINED OUTLINE OF ESSENTIAL DATA REQUIRED FOR A REPORT ON A PLANT OR A PROJECT

(*d*) General map showing lands owned, leased, or under option, and required for extension or new work, and location of present and proposed structures.
(*e*) Contour map and sections at sites of all structures showing foundation conditions.
(*f*) Profile of river.
(*g*) Get if possible
(1) U. S. G. S. maps.
(2) U. S. Coast and Geodetic Survey maps.
(3) Other national state or municipal maps.
(4) Maps of private surveys.
2. Photos: get photos of present structures and sites of proposed structures.
3. Drawings: get blueprints of all present and proposed structures.
4. Literature: get copies of all available reports.

VIII. Legal.
1. Has the power company the right under the local law to condemn
(*a*) The flow of the streams?
(*b*) The necessary real estate?
2. How will water rights be acquired?
3. How will lands and rights of way be secured?
4. What approval of city, state, or national government is necessary for the proposed extension and new work?
5. What legislation is necessary?
6. What parts of items 4 and 5 have been secured?
7. How long will it take to secure such approval and legislation?
8. Are fishways required?
9. Will log chutes be required?
10. Can transmission-line right of way be secured on sectional lines or public highways free of charge?
11. What will taxes on proposed extensions and new work be?
12. What will the taxes on existing work, bonds, and stocks be?
13. What will the political aspects be?

IX. Power available.
1. Kw-hr per annum of primary output.
2. Kw-hr per annum of secondary output in average year.
3. Kw-hr per annum of secondary output in minimum year.
4. Kw-hr per annum of dump output in average year
5. Kw-hr per annum of dump output in minimum year.
6. Distribution of foregoing items during week and year.
7. Load factor required for delivered power.

X. Power markets.
1. Communities being or to be served.
2. Existing and prospective power contracts.

TABLE 4—*Continued*

COMBINED OUTLINE OF ESSENTIAL DATA REQUIRED FOR A REPORT ON A PLANT OR A PROJECT

3. Large power users.
4. Street lighting.
5. Domestic users.
6. Load curves.
7. Load factors.
8. Probable growth of market.
9. Probable sale price of power or existing rates.
10. Probable competition.
11. Probable cost of steam power for the market.

XI. Description of existing physical property.
1. Lands.
   (*a*) Location.
   (*b*) Description.
   (*c*) Areas.
   (*d*) Detailed actual or estimated value.
   (*e*) Used for what purpose at present.
   (*f*) Owned in fee.
   (*g*) Leased (description of lease).
2. Water rights.
   (*a*) Location.
   (*b*) Nature.
   (*c*) Detailed actual or estimated value
   (*d*) Owned.
   (*e*) Leased (description of lease).
   (*f*) Controlled by contract (description of contract).
3. Structures and apparatus.
   (*a*) Itemized list (see Table 2).
   (*b*) Description, general dimensions, capacities, etc.
   (*c*) Drawings.
   (*d*) Character of foundations.
   (*e*) Age, condition, and probable life.
   (*f*) Detailed actual or estimated cost.
   (*g*) Provisions for extensions.
   (*h*) Owned.
   (*i*) Leased (description of lease).
4. Reservoirs and ponds.
   (*a*) Areas.
   (*b*) Permissible draft.
   (*c*) Capacity.
   (*d*) High water.
   (*e*) Relocated roads, bridges, railroads, etc.

XII. Proposed new work and extensions.
1. Lands now owned or leased (see XI, 1).
2. New lands required.

TABLE 4—*Continued*

COMBINED OUTLINE OF ESSENTIAL DATA REQUIRED FOR A REPORT ON A PLANT OR A PROJECT

(*a*) When needed.
(*b*) Area.
(*c*) Location.
(*d*) Required for what purpose.
(*e*) Used for what purpose at present.
(*f*) Under option (description of option).
(*g*) Otherwise controlled.
(*h*) Not controlled.
(*i*) Detailed probable cost.

3. Water rights, now owned or under contract (see XI, 2).
4. Additional water rights required.
   (*a*) Location and when needed.
   (*b*) Nature.
   (*c*) Under option (description of option).
   (*d*) Otherwise controlled.
   (*e*) Not controlled.
   (*f*) Detailed probable cost.
5. Proposed new construction.
   (*a*) When needed.
   (*b*) Itemized list (see Table 2).
   (*c*) Description, general dimensions, capacities, etc.
   (*d*) Sketches or drawings.
   (*e*) Description of subsurface investigations.
   (*f*) Character of foundations.
   (*g*) Detailed estimate of construction cost.
6. Reservoirs and ponds to be created.
   (*a*) Areas.
   (*b*) Permissible draft.
   (*c*) Capacity.

XIII. Data for estimates of cost.
1. Kind and location of material for construction.
   (*a*) Ingredients for concrete.
   (*b*) Stone for riprap.
   (*c*) Earth for embankments.
   (*d*) Timberlands.
   (*e*) Are foregoing on owned or controlled lands or lands to be purchased?
2. Local costs.
   (*a*) Skilled labor.
   (*b*) Unskilled labor.
   (*c*) Trucks.
   (*d*) Coal.
   (*e*) Lumber.
   (*f*) Other materials.
3. Available camp sites.
4. Freight rates.

TABLE 4—*Continued*

COMBINED OUTLINE OF ESSENTIAL DATA REQUIRED FOR A REPORT ON A PLANT OR A PROJECT

5. Local hauling rates.
6. Local rates for power for construction purposes.
7. Storage reservoir assessments.

XIV. Operation of existing properties.
1. Statement of assets and liabilities for a period of 5 years.
   (*a*) General balance sheet
   (*b*) Nature of funded debt.
      (1) Interest rates and dates.
      (2) Date of maturity of bonds
      (3) Premium and date at which bonds may be redeemed.
   (*c*) Nature of floating debt.
   (*d*) Contingent liabilities (i.e., cumulative dividends, etc.).
   (*e*) Debts not due by ascertained amounts (i.e., taxes, sinking fund, depreciation, suits pending, adverse legislation, etc.).
   (*f*) Profit and loss account.
   (*g*) Schedule of construction costs.
2. Statements of receipts (for period of 5 years).
   (*a*) Lighting.
      (1) Commercial.
      (2) Municipal
   (*b*) Power.
      (1) Commercial.
      (2) Municipal.
   (*c*) Other sources.
3 Statements of operating expenses (for a period of 5 years).
   (*a*) Manufacturing.
      (1) Fuel, oil, waste, and supplies.
      (2) Labor.
      (3) Maintenance.
   (*b*) Distribution.
      (1) Labor.
      (2) Maintenance.
   (*c*) General.
      (1) Office salaries.
      (2) Supplies.
      (3) Rents.
      (4) Taxes.
      (5) Legal.
      (6) Insurance.
      (7) Damages.
4. Statement of net earnings (for period of 5 years).
5. Statement of deductions (for a period of 5 years)
   (*a*) Fixed charges.
   (*b*) All other.
6. Statement of the surplus (for a period of 5 years).

TABLE 4—*Continued*

COMBINED OUTLINE OF ESSENTIAL DATA REQUIRED FOR A REPORT ON A PLANT OR A PROJECT

7. Statement of special cause for material increase or decrease in earnings and operating expenses during any year.
8. Statistics (for a period of 5 years).
   (*a*) Power-station load curves showing peaks and load factor.
   (*b*) Cost per kw-hr generated (i.e., switchboard cost).
   (*c*) Cost of power per kw-hr, if purchased, and contract conditions.
   (*d*) Gross receipts per kw-hr.
   (*e*) Gross receipts per capita served.
   (*f*) Percentage of operating expense to gross receipts.
   (*g*) Water consumption and cost.
   (*h*) Kw-hr generated per annum.
   (*i*) Output of substations.
   (*j*) Kind of fuel, consumption, and cost delivered.
9. Fund for depreciation.

XV. Possible competition.
1. By existing competitors.
2. By possible future competitors.
3. Cost of steam power.
4. Cost of coal.

The engineer's detailed office studies and calculations generally remain in his possession. The need in these for careful and complete compilation of all assumptions, data, and calculations, properly indexed, logically arranged, and cross-referenced, cannot be overemphasized. The basic data in all sections of the calculations should give the source from which they were obtained or a reference thereto, and all studies should be explained to an extent sufficient to allow another engineer to follow them. The engineer should bear in mind the possibility that, after several years, he may be called upon to expand or explain parts of the report and perhaps to testify in a legal action in connection with the subject of the report.

The writer has found the form given in Table 3 useful for compiling a summary of data for quick reference. The form is particularly well adapted to the compilation of data for the various projects of a large system.

Table 4 gives an outline of the essential data required for a report on a plant or a project. A list of this kind is useful as a guard against omissions in field investigations and in the compilation of the report.

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DAMS—DESIGN AND CONSTRUCTION

CHAPTER 17

SOLID GRAVITY CONCRETE DAMS

A. General

1. Introduction. An extensive discussion of the theory of the design of dams is not possible in a handbook. Therefore, this volume is confined to a synopsis of the subject and the presentation of sufficient data for the engineer to prepare preliminary designs and estimates. For a complete treatise on the design of dams the reader may consult Refs. 1 and 10, Section 57.

2. The Choice of Solid Gravity Concrete Dams. The solid gravity concrete dam is a permanent structure. Past experience has indicated that the annual maintenance cost for this type of dam is greater than for rock-fill or earth dams and less than for any other type However, improvements in modern practice of concrete materials and construction will tend to make future maintenance costs negligible.

This type of dam is adaptable to all localities, but its height is limited by the strength of the foundations, the height of those on earth foundations having been limited generally to about 65 ft. For localities where the rock is a considerable distance below the surface, an earth-fill dam has frequently been found to be more economical, particularly when a dam of great height is required, because the earth-fill dam does not have to rest on a rock foundation.

The difference in first costs between solid and hollow gravity concrete dams depends on local conditions. The solid dam requires less cement per cubic yard of concrete, less form work, and less expense in placing concrete, and it has no steel reinforcement. On the other hand, the hollow dam requires considerably less concrete per linear foot of dam. The lightest type of hollow dam usually requires only 35 to 40% of the concrete required in a solid dam. For a remote location, where materials of construction are expensive, the hollow dam usually costs less to build than the solid dam; but, in more accessible locations, comparatively near railroads, where concrete aggregates are convenient, the reverse is likely to be true.

The solid gravity concrete dam usually costs more than a timber dam, though perhaps not more than a first-class, rock-filled, timber crib dam at a site where timber is expensive.

An earth or rock embankment almost always costs considerably less than any form of gravity concrete dam, if materials for the former are found convenient to the site. Therefore, if conditions admit of an embankment, that

type of dam is usually to be preferred. The limitations of earth dams are discussed in Section 3, Chapter 20.

Because there is considerably less material in an arch dam than in any other concrete type, it costs much less to construct. However, as will be pointed out later, sites particularly suitable for arched dams are rare.

3. Nomenclature. The following nomenclature will apply to the design of solid gravity concrete dams. Unless definitely mentioned, all forces are stated in pounds and all dimensions in feet. These symbols are also defined where they first appear.

- A = area of a vertical or horizontal section of the dam; angle between wind direction and fetch
- a = earthquake acceleration = αg.
- C_e = an earthquake factor.
- D = average depth of water in setup equations.
- d = a horizontal length, with locally described subscripts.
- e = eccentricity, distance from the center of gravity of a section to its midpoint.
- F = fetch or length in miles of exposure of a water surface to wind action.
- f = coefficient of static friction for well-dressed specimens.
- f' = actual coefficient of static friction at a given joint or base.
- g = acceleration of gravity = approximately 32.2 ft per second per second.
- H = total height of dam section.
- h = vertical distance; height of masonry; head of water, etc.; special subscripts explained where introduced.
- h_r' = "design head" on standard spillway crest.
- h_w = height of waves; head on theoretical sharp-crested weir.
- k = percentage of voids expressed as a decimal.
- L = top width of a dam
- l = length of a horizontal joint.
- M = mass; moment, special subscripts explained where introduced.
- ΣM = algebraic summation of the moments of all forces acting above a given joint or base, including uplift but exclusive of the reaction at the joint or base.
- m = distance from the center of gravity of a figure or base of a dam block; m' is to the downstream face; m'' is to the upstream face.
- P = horizontal load on a gravity dam; frequently divided into P_1, P_2, etc.
- ΣP = algebraic summation of the horizontal components of all forces acting on a gravity dam above a given joint or base, excluding the reaction at the joint or base.
- P_e = an earthquake force.
- P_i = ice pressure per linear foot of dam.
- P_s = horizontal earth or silt pressure.
- p_i = inclined unit stress; p_i' is at the downstream face; p_i'' is at the upstream face.
- p_n = external normal unit pressure due to water and silt on the face of the dam; p_n' is on the downstream face; p_n'' is on the upstream face.
- p_r = unit vertical reaction at a point in the foundation of a dam exclusive of uplift pressure; p_r' is at the downstream toe; p_r'' is at the upstream toe
- p_u = unit uplift or piezometer pressure on a joint or base of a dam; p_u' is at the downstream toe; p_u'' is at the upstream toe.
- p_v = total unit vertical reaction $(p_r + p_u)$; p_v' is at the downstream toe; p_v'' is at the upstream toe.
- R = a reaction or a resultant of forces.
- r = ratio of average to maximum shearing stresses at the joint or base.
- S = setup caused by wind.
- S_f = safety factor against sliding.

$S_{s\,f}$ = shear-friction safety factor.
s_u = ultimate unit shearing strength of dam or foundation materials.
V = wind velocity in miles per hour.
W = vertical force or weight; subscripts designate special values.
ΣW = algebraic summation of the vertical components of all forces acting on a dam above a given joint or base including uplift but exclusive of the reaction at the joint or base.
W_u = total uplift force in a dam slice at a joint or the foundation.
w = unit weight.
w_1 = unit weight of masonry.
w_2 = unit weight of water.
w_3 = unit dry weight of earth or silt.
w_4 = unit weight of submerged earth or silt.
w_5 = unit weight of saturated earth or silt.
x = in general a distance; a vertical or horizontal lever arm.
y = in general a distance.
z = horizontal distance from center of moments to the point of intersection of the resultant with a joint or base.
α = ratio of earthquake acceleration to g.
ζ = uplift intensity factor.
θ = angle of inclination * with the vertical of the resultant R of the forces $\Sigma(W)$ and $\Sigma(P)$.
λ = specific gravity.
ϕ = angle of inclination * with the vertical of the face of the dam; ϕ' is for the downstream face; ϕ'' is for the upstream face; angle of internal friction for earth or silt.

B. Forces Acting on Dams

4. The Forces Considered in Design. In the design of any dam the forces generally considered as acting on the structure may consist of the following:

a. Water pressure.	*e*. Weight of the dam.
b. Atmospheric pressure.	*f*. Weight of the foundation.
c. Ice pressure.	*g*. Earthquake forces.
d. Earth pressure.	*h*. Reaction of the foundation.

The nature of most of these forces, unfortunately, is such that they do not admit of exact determination. Their amounts, direction, and location must be adopted by the designer after a thorough consideration of all obtainable facts bearing on the case, and with the exercise of his best judgment, based on his experience and that of others who have had to deal with similar problems. It must always be borne in mind that conditions in no two dams are alike, and that a general theory must never be applied to a particular case without thought as to the possible need of modification to suit the conditions peculiar thereto.

5. External Water Pressure. The weight of a cubic foot of fresh water usually adopted in dam design is 62.5 lb per cu ft.

* This is the common definition, as in general the joints and bases are horizontal. For inclined joints or bases the angles θ and ϕ should be measured from a normal to the joint or base.

In Fig. 1, let 1–2 represent a submerged vertical rectangular plane of unit width, measured perpendicular to the paper, having its top edge parallel to and coincident with the surface of the water.

The total pressure, P, on the plane 1–2 is obtained by the equation

$$P = \tfrac{1}{2}w_2h_2^2 \qquad [1]$$

The force, P, is a distributed force, applied, although unequally, over the entire face. It may be represented by the right triangle 1–2–3, the length of

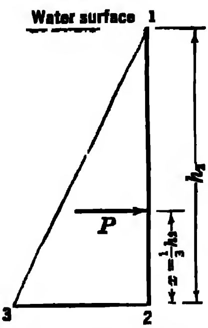


FIG. 1 Triangular water pressure on a vertical plane

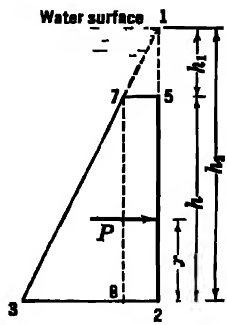


FIG. 2 Trapezoidal water pressure on a vertical plane

the leg 2–3 of which is proportional to w_2h_2. Its center of application passes through the center of gravity of the triangle, which is up from the bottom of the face a distance

$$x = \tfrac{1}{3}h_2 \qquad [2]$$

The moment of P about the lower edge of the surface is

$$Px = \tfrac{1}{6}w_2h_2^3 \qquad [3]$$

If a portion 1–5 of the face 1–2 is removed, as indicated in Fig. 2 for a spillway dam without velocity of approach, there will be left a vertical submerged face 5–2 of length h, with its upper edge below water surface a depth h_1. The total pressure on the surface 5–2 is

$$P = \tfrac{1}{2}w_2(h_2^2 - h_1^2) \qquad [4]$$

The center of application passes through the center of gravity of the trapezoid 5–2–3–7, which may be found graphically or by the equation

$$x = \tfrac{1}{3}h\,\frac{h_2 + 2h_1}{h_2 + h_1} \qquad [5]$$

The moment of the force P about the lower edge, 2, of the surface is

$$Px = \tfrac{1}{2}w_2h^2(h_1 + \tfrac{1}{3}h) \qquad [6]$$

In the case of a spillway dam with velocity of approach v, as shown in Fig. 3, the water surface, which at the non-overflow portion of the dam would be represented by the line 1–8, will drop a distance $h_v = v^2/2g$ to level 9–10 at the spillway and 1–8 becomes the energy gradient. The pressure on the dam, including the dynamic pressure due to the velocity of approach, corresponds to the elevation of the energy gradient and not the elevation of water surface. The total pressure, P, on the face of the dam 5–2 is represented by the area 5–4–3–2. For practical purposes the total pressure may be assumed to be equal to the area 5–7–3–2, and Eqs. 4, 5, and 6 will apply.

The submerged faces of dams are frequently inclined, causing the normal water pressures against them to depart from the horizontal. In most dam-

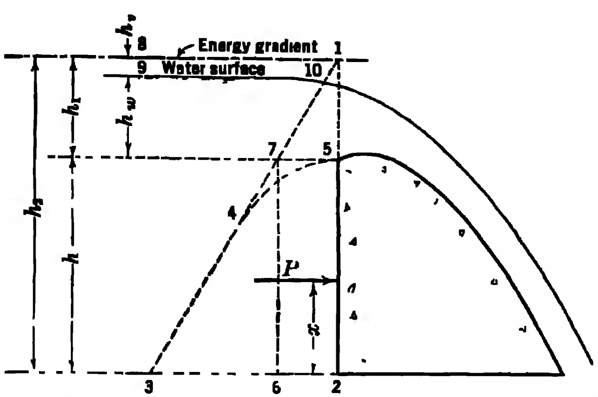


FIG. 3. Trapezoidal water pressure on a vertical face of an overflow dam.

design problems, it is convenient to deal separately with the horizontal and vertical components of such forces.

If the face 5–2 of the spillway dam of Fig. 3 is now considered inclined, as the face 1–2 in Fig. 4, then the total pressure R on the face 4-2 may be resolved into horizontal and vertical components, P and W. The horizontal component P will be equal to the pressure represented by the area 5–7–3–2 on the projection 2–5 of the plane 1–2, and its amount, center of application, and moment about point 2 can be computed by Eqs. 4, 5, and 6. The vertical component, W, including the vertical component of the dynamic pressure due to the velocity of approach, will be equal to the weight of water represented by the area 1–2–4–8 with its center of application at the center of gravity of the figure.

In Figs. 3 and 4, if h_1 becomes equal to zero, as for a case of no flow, the spillway dam is similar to a non-overflow dam. The energy gradient and water surface are coincident. With h_1 equal to zero, Eqs. 4, 5, and 6 apply for a non-overflow dam.

The horizontal component, P, of the total water pressure on the upstream face of the dam in Fig. 5 is equal to the total pressure on the plane 8–3 and

is determined by Eq. 1. The distance, x, from point 3 to the center of application of the force P is found from Eq. 2. The vertical component, W, is equal to the weight of water within the area 7–2–3–8 and acts through the center of gravity of that area. For convenience, W may be divided into the parts 7–2–9–8 and 2–3–9. The tailwater of depth h_4 exerts a pressure in an

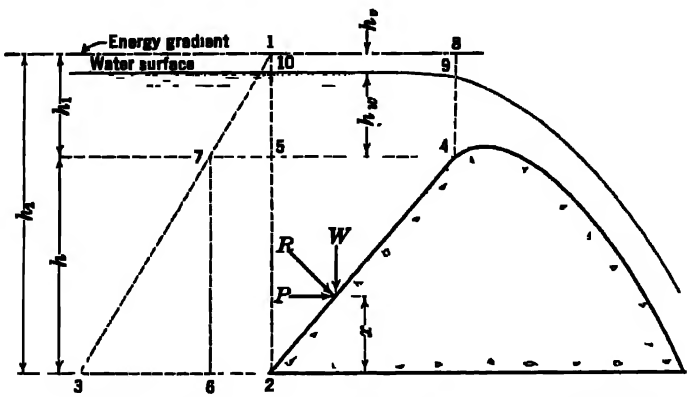


FIG. 4. Water pressure on an inclined face of an overflow dam.

upstream direction, the value and center of application being determined respectively by Eqs. 1 and 2. The vertical tailwater effect is equal to the weight within the triangle 11–10–4.

The presence of tailwater may reduce or may increase the stability of the dam. If the correct tailwater depth is difficult to determine, the stability of the dam should be checked for possible maximum and minimum depths.

In the case of a spillway section, consideration should be given to the possible reduction in depth due to a hydraulic jump.

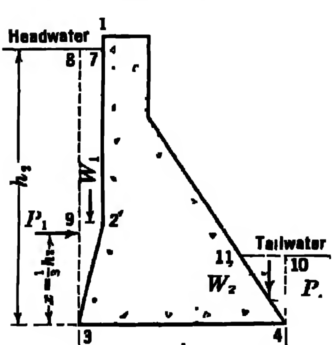


FIG. 5. Water loads on a gravity dam.

6. Subatmospheric Effects. In Fig. 3, if the shape of the crest of the spillway is made to fit the jet of spilling water, as explained in Section 33, the falling water will not exert any appreciable force on the dam. However, with a crest not shaped to fit the jet, and with insufficient aeration under the jet, a partial vacuum is created under the jet. Such an area of subatmospheric pressure along the face of the dam adds to the overturning effect on the dam.

7. Uplift. Dams are subjected to water pressure, not only on exposed faces but also on their bases and within the masonry itself. These internal pressures

produce uplift. Uplift is the upward piezometric pressure of water as it seeps or flows through the dam or its foundations. It causes a reduction in the effective weight of the structure above it. Water causing uplift pressures may enter through pores or imperfections in the foundation, through imperfectly bonded foundation or construction joints, or through pores in the structure itself. The conception generally applied to hydrostatic-uplift pressure may be explained by a discussion of the hypothetical conditions illustrated in Fig. 6a. This figure represents a dam on a rock foundation with an upstream water depth of h_2 and a tailwater depth of h_4. Consider the base as slightly

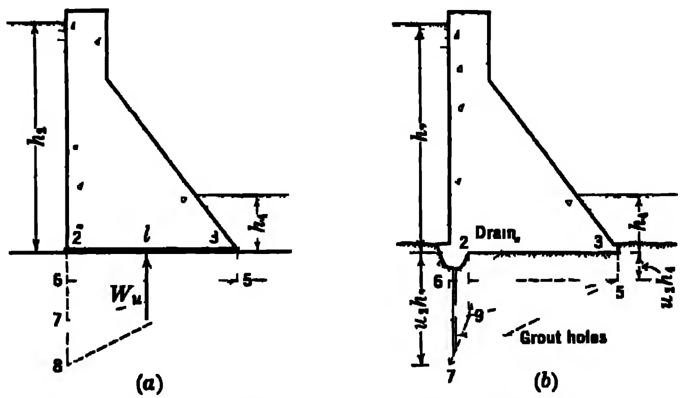


Fig. 6 Uplift pressure intensity diagrams

raised from the foundation permitting flow from the upper to the lower pool. Assuming that the width of crack is uniform, that the flow conforms to the laws governing flow in pipes, and neglecting loss at entrance, the pressure intensity or piezometric pressure corresponding to the hydraulic gradient will diminish uniformly from w_2h_2 at point 2, as represented by line 2–8, to w_2h_4 at point 3, as represented by line 3–5.

Suppose that without lowering the dam the crack is completely closed at point 2, thus stopping the flow. The piezometric pressure now becomes constant as shown by line 6–5 and equal to w_2h_4.

If the closure at point 2 is only partially effective, the pressure intensity at point 2 is represented by the line 2–7, or

$$p_2 = w_2[h_4 + \xi(h_2 - h_4)] \qquad [7]$$

the pressure intensity at point 3 is represented by the line 3–5, or

$$p_3 = w_2h_4 \qquad [7a]$$

and the total uplift pressure which is represented by the pressure intensity diagram 2 7 5 3 is given by the equation

$$W_u = w_2[h_4 + \tfrac{1}{2}\xi(h_2 - h_4)]l \qquad [7b]$$

where p_u'' = the uplift pressure intensity at the upstream toe;
p_u' = the uplift pressure intensity at the downstream toe;
W'_u = the total uplift pressure;
ζ = the uplift intensity factor—the proportion of net head not dissipated by the cutoff;
A = the area of the base of the dam which, for a 1-ft length of dam, is equal to the distance 2–3.

The line of application of the total uplift force passes through the center of gravity of the pressure-intensity diagram.

Actually, the dam of Fig. 6a is not raised from its foundation, but water is forced through the pores and cracks of the masonry and the foundation materials. Water flowing through such pores or cracks follows a similar law of decreasing pressures.

For important high dams on rock, a grout-curtain cutoff is frequently provided near the upstream toe of the dam supplemented by drilled holes immediately downstream from the grout curtain, as indicated by Figs. 6b and 19, to act as a drainage system. This drainage system supplements the grout curtain to reduce uplift. In Fig. 6b, the effect of the drainage system is to reduce the pressure intensity diagram from 2–7–5–3 to 2–7–9–5–3. In practice the drainage holes are near enough to the grout curtain so that the uplift diagram may be assumed to be 2–4–9–5–3. Tests on existing dams, with grout curtain and drainage, have indicated that values of the uplift pressure intensity factor, ζ, have not exceeded ⅔, many showed values less than ½, and, with careful work, some have been close to zero.

The claim has been made that, for dams on rock, only a percentage of any horizontal area is subjected to uplift pressures. However, this has not been proved to be true, unless the area considered is capable of resisting tensile stresses. It is recommended that 100% of the area be assumed as subjected to uplift pressures corresponding to whatever piezometric hydraulic gradient, line 7–5 in Fig. 6, is used in the design.

In the case of dams on rock which are not provided with a grouted cutoff or drainage, the value of ζ would theoretically be unity and the uplift pressure intensity diagram would correspond to the full hydraulic gradient between headwater pressure and tailwater pressure. However, for low and medium-height dams, with massive rock foundations, standard practice does not require ζ to be unity.

For a well-built concrete dam on a massive rock foundation it can be assumed that the foundation, the junction of the dam with the foundation, and the dam itself can have a moderate tensile strength of at least 85 to 130 lb per sq in. This tensile strength is sufficient to support an uplift pressure of about 200 to 300 ft of water. In this respect, standard practice has tacitly admitted the reliability of tensile stresses adding to the stability of the dam, contrary to the provisions of Rule 1 of design given in Section 18.

Thus it would seem that all dams on rock foundations and of heights above about 200 to 300 ft should be provided with drainage both in the foundation and in the dam or ζ should be assumed to be unity.

A silt deposit which has a tightness equal to or greater than that of the foundation will reduce the total forces acting on the dam [2], because it causes a reduction in water pressure on the upstream face and the base, the effect of which is greater than the effect of the silt pressure itself.

However, in dams on tight rock foundations, the silt must be assumed to be much more permeable than the foundation, and its effect on upstream face water pressure and underpressure should be neglected.

For preliminary designs on rock foundations, and without previous knowledge of what system of grout curtain and drainage will be adopted, the following values of ζ are recommended:

Height [1] of Dam	Type of Rock Foundation	Grouting [2] and Drainage	ζ
Medium	Horizontally stratified	None	1.00
Medium	Fair, horizontally stratified	Yes	0.67
High	Fair, horizontally stratified	Yes	0.75
Medium	Good, horizontally stratified	Yes	0.50
High	Good, horizontally stratified	Yes	0.67
Medium	Fair, massive	None	0.67
Medium	Fair, massive	Yes	0.50
High	Fair, massive	Yes	0.67
Medium	Good, massive	None	0.50
Medium	Good, massive	Yes	0.50 [3]
High	Good, massive	Yes	0.50

[1] "Medium" represents dams up to about 200 to 300 ft. "High" represents dams above about 200 to 300 ft.

[2] Assumed to be first-class.

[3] A minimum limit.

For dams on earth foundations without a cutoff or drainage, Eq. 7b applies approximately, the actual uplift pressure being as indicated in Fig. 7 of Chapter 25, the value of ζ being, of course, unity, and the full area of the base being subjected to the hydrostatic pressure. Figure 7 of Chapter 25 indicates also the hydrostatic gradient for various depths of cutoffs in infinitely thick earth foundations without drains. If the cutoff extends to impervious materials, the hydrostatic pressure will be greatly reduced, with a value of ζ equal to zero as a limit.

8. Ice Pressure. Edwin Rose [3] makes the following recommendations for the determination of the thrust to be expected from an ice sheet. In Fig. 7 the thrust is given in terms of air-temperature rise, ice thickness, and lateral restraint.

Curves A, B, and C are to be used for air-temperature changes at rates of +5, +10, and +15 degrees Fahrenheit per hour respectively. All curves

are based on an assumed initial air temperature of $-40°$ F, and an initial ice temperature varying linearly from $-40°$ F at the top surface to $+32°$ F at bottom surface. Air temperatures are assumed to rise at the indicated rates until a temperature of $+32°$ F is reached, then to remain constant at $+32°$ F until the temperature throughout the ice is about $+32°$ F.

The maximum rate of temperature rise can be obtained from meteorological records for the region. A sustained temperature rise for several hours at a rate approaching 5° F per hr might reasonably be expected in almost any region of subzero winter temperature. Rates approaching 15° F per hr are

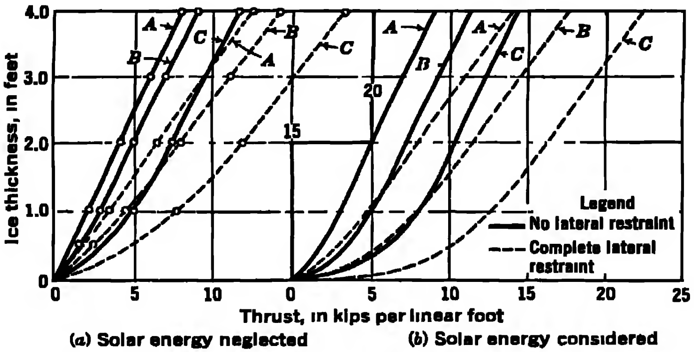


FIG. 7. Ice thrusts for the variables, ice thickness, air-temperature rise, and restraint. (Edwin Rose, "Thrust Exerted by Expanding Ice Sheet," *Proc. A.S.C.E.*, May 1946, p. 574.)

not uncommon where chinook winds occur, as along the eastern slope of the Rockies and parts of the Pacific Northwest.

The maximum thickness of ice sheet to be expected can be determined from historical data for the region. Thicknesses in the United States seldom exceed 2 or 3 ft.

The extent of restraint can be estimated by study of the topography and of the location and type of structure involved. For example, if the bank or wall parallel to the axis of a straight dam or hydraulic structure is steep, if the banks or walls on the sides are also steep and sound, and if the ice is solidly frozen to the restraining edges, then restraint is complete or nearly so. Where the horizontal dimensions of ice sheet between the restraining walls or banks are not large, conditions more favorable to higher pressures may exist, owing to the likelihood of fewer cracks and consequent better continuity and also to the greater stability against buckling than for large ice sheets. If the opposite bank or wall is steep but the banks on the sides are sloping or unstable, the condition of no lateral restraint is approached. If the edge of the ice sheet on all sides is free to slide up sloping banks, then a condition of no restraint exists; consequently, the pressure exerted against the structure is

minor. The curves for complete lateral restraint in Fig. 7 are based on the very severe condition of elastic restraint and, as such, represent the extreme pressures for the air-temperature change and thickness considered. After estimating the restraint conditions, design values can be interpolated on Fig. 7 for specific values of temperature change and thickness.

Figure 7*a* indicates the condition for solar energy neglected. Direct exposure of the ice sheet to the sun's rays results in an additional ice-temperature rise and the transmission of increased thrust to the dam, as indicated in Fig. 7*b*, for solar energy considered. If the upstream face of the dam is in the shade so that the ice sheet is likely to be well frozen to the dam and if a similar condition exists around the shore near the dam, but if a considerable area of the central part of the ice is exposed to the sun's radiation, then a condition probably exists favorable to the development of an increase in pressure against the dam as a result of solar energy absorption.

Where the dam is provided with an overflow spillway, the spillway crest is usually some distance below the maximum or design water level. The maximum water level occurs only at times of freshet, and a solidly frozen ice sheet at such time is improbable. It is usual to assume that the worst ice condition will occur only with water at the spillway lip.

Nothing is known of the action of ice during an earthquake, and its earthquake effect is ignored.

9. Earth and Silt Pressures. (*a*) *Source.* Concrete dams are sometimes subjected to earth pressures on either the downstream or upstream face, where the foundation trench is backfilled. Such pressures usually have a minor effect on the stability of the structure and may be ignored in design.

Practically all streams transport silt, particularly during floods when the quantities may be enormous. Such material is deposited in the reservoirs or slack water above the dam and should be taken into consideration unless provision is made for its removal. Quite often, sluices are constructed in the lower part of the dam which, if periodically flushed in the proper manner, limit the depth of such deposits adjacent to the dam. However, the continued operation of such sluices may be neglected.

Pressures on the dam when the reservoir is full correspond to the effective pressure due to submerged silt. When the reservoir is empty, they may be taken as due to saturated silt.

(*b*) *Theoretical Pressure.* A widely used formula for silt pressure is the following one attributed to Rankine:

$$P_s = \frac{wh_s^2}{2}\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right) \qquad [8]$$

where P_s = the total horizontal pressure in pounds;
w = the weight of silt in pounds per cubic foot;
h_s = the depth of silt in feet;
ϕ = the angle of internal friction of the material.

The force P_s is assumed to be horizontal and located a distance $2h_3/3$ below the surface of the silt. This equation assumes a level fill, the usual condition for silt or water-borne detritus, and ignores the friction between the silt and the wall. Experiments indicate that ϕ is not changed materially by submergence; that silt pressure and water pressure exist coincidentally in a submerged fill [4]; and that the pressure exerted by the submerged silt, over and above water pressure, is reduced in the proportion that the weight of the silt is reduced by submergence.

If the face against which the silt acts is inclined, the weight of the silt directly above the inclined face may be included in the vertical forces acting on the dam.

(c) *Computation of Submerged Weight.* The dry weight, w_3, can be estimated from samples of silt deposits in neighboring reservoirs. The unit weight of submerged silt is

$$w_4 = w_3 - w_2(1 - k) = w_3\left(\frac{\lambda - 1}{\lambda}\right) \qquad [8a]$$

where w_4 = the submerged unit weight of silt in pounds per cubic foot;
w_3 = the unit dry weight in pounds per cubic foot;
k = the percentage of voids, expressed as a decimal;
λ = the specific gravity of the solid particles.

(d) *Computation of Saturated Weight.* The unit weight of saturated silt is

$$w_5 = w_3 + kw_2 = w_2 + w_3\left(\frac{\lambda - 1}{\lambda}\right) \qquad [8b]$$

(e) *Values of Constants.* Values commonly used for the constants in the foregoing equations are as follows:

w_3 = dry weight = 100 lb per cu ft;
k = voids = 40%;
λ = specific gravity = 2.67.

Experimental values for the angle of friction, ϕ, are scarce. Where silt pressure is of primary importance in the design of a dam, a special study including experimental work may be required. For simple conditions, where silt pressures require consideration but are not so important as to demand great accuracy, the value of ϕ may be taken as about 30 degrees.

10. Wave Heights. The upper portions of dams are subject to the wave action. The dimensions of waves depend on the extent of the water surface and the velocity of the wind, among other factors.

Knowledge of wave heights is important if overtopping by waves is to be avoided. Formulas for wave heights proposed by Stevenson have been widely used. Molitor [5] proposes modifications of the Stevenson formulas to include the wind velocity, as follows:

$$h_w = 0.17\sqrt{VF} + 2.5 - \sqrt[4]{F} \qquad [9]$$

where h_w = the height of the wave from trough to crest, in feet;
V = the wind velocity in miles per hour;
F = the "fetch" or straight length of water, in statute miles, subject to wind action.

For F greater than 20 mi, this equation may be simplified thus:

$$h_w = 0.17\sqrt{VF} \qquad [10]$$

Equation 9 gives a wave height of 2.5 ft when the wind velocity is zero, which is not correct.

The authors propose the following equation, which, though more rational in this respect than Eq. 9, gives practically identical results.

$$h_w = \frac{F^{0.37}V^{0.48}}{3.41} \qquad [11]$$

It is assumed that the wave will ride up a vertical slope or a smooth inclined face a vertical distance above still water equal to $1.5h_w$. On rough inclined faces, such as dumped rock riprap on earth dams, the ride up will be less, and a value of $1.4h_w$ has been used.

The increased height of water surface, due to waves, is not included in the forces acting on the dam.

11. Tides, Setup, and Seiches. Tide movements are imperceptible in inland waters, the maximum for the Great Lakes being less than 1 in. [6]. However, an appreciable piling up of water on one shore of a lake or reservoir may be caused by wind action, particularly in shallow water. The height of rise above the undisturbed lake level is called "setup." For deep water and small areas, this effect is small and may be considered included in the freeboard allowance. For long, shallow reservoirs a special study may be required. The Zuider Zee formula [7] is the best available means of estimating setup. It is as follows:

$$S = \frac{V^2F}{1400D}\cos A \qquad [12]$$

where S = the setup in feet above still pool level.
V = the wind velocity in miles per hour;
F = the fetch in miles;
D = the average depth of water in feet;
A = the angle of wind and fetch.

The setup should always be added to the estimated elevation of flood water surface

Periodic undulations, called seiches, also occur. Seiches may be set in motion by intermittent wind action, variations in atmospheric pressures, earthquakes, or irregular inflow or outflow. They come and go at regular periods that may vary from a few minutes to several hours. After the gen-

erating influence is removed, the oscillations gradually subside. Amplitudes of 0.5 ft or more can readily occur in reservoirs of moderate size, but no information is available for computing their magnitude. The factor of safety used in computing freeboard should be sufficient to cover possible seiches.

12. The Weight of the Dam. Construction of any important concrete dam involves a careful analysis of available concrete materials, that should be based on an adequate treatise on concrete. Such an analysis includes data on weights. It is frequently necessary to proceed with designs, at least in a preliminary manner, before a complete concrete analysis is available.

In the absence of exact information, 150 lb per cu ft for the design weight of concrete will be found to conform to modern practice.

13. The Weight of the Foundation. Dams have sometimes been tied down to the rock foundation in order to increase their resistance to overturning, sliding, or both. Steel bars or cables are grouted into holes bored in the rock and extended into the dam near the upstream face. This practice has been criticized on the ground that a satisfactory anchorage of the bars in the foundation is seldom possible.

14. Earthquake Forces. (*a*) *General Statement.* In regions where earthquakes occur, dams must resist the inertia effects caused by the sudden movements of the earth's crust. If the foundation under a dam moves, the dam must move with it, if rupture is to be avoided. To produce such motion, forces to overcome the inertia of the structure and its loading must be applied through the medium of stresses in the dam and its foundation. The magnitude of such stresses is determined primarily by the intensity of the earthquake and by the effect of the earthquake on the mass of the structure and its loading. The effect of resonance, which would be considered in a stress analysis of the earthquake effect in buildings, towers, etc., is almost invariably ignored for dams.

Dams have a very favorable record in resisting earthquake shocks. Nevertheless, it must be true that earthquake forces reduce the factor of safety, and conservatism demands their full consideration in seismically active regions. There are few if any regions where small earthquake disturbances certainly can be said to be improbable.

Earthquake forces, of course, must be combined with other forces acting on the dam. Because of their short duration and infrequent occurrence, some concession in factor of safety is permissible for the assumption of maximum earthquake and the most adverse combination of other conditions. For open-spillway dams, earthquake and maximum flood frequently are not assumed to occur simultaneously.

(*b*) *Intensity.* The intensity of the inertia force depends on the acceleration, i.e., on the rate of change in the velocity of motion. This acceleration, a, is usually designated by its ratio to g, the acceleration of gravity, or $a = \alpha g$.

Most dams in seismically active regions of the United States have been designed for an acceleration of one-tenth gravity, or $a = 0.1g$. The values of $\alpha = 0.1$ for alluvial valleys and 0.05 for dense rock may be considered tentatively

standard for dams in seismically active regions. For sites close to known active faults, larger values should be adopted. In favorable locations, smaller values can be justified. The designer should carefully study local conditions and particularly the seismographical history of the region. For important structures, an investigation should be made by a competent geologist.

(c) *Inertia of Masonry.* The force required to accelerate a given mass, such as the body of a dam, is found from the equation

$$P_e = Ma = \frac{W\alpha g}{g} = \alpha W \qquad [13]$$

where P_e = the horizontal earthquake force;
M = the mass of the dam, or any portion of it under consideration;
a = the earthquake acceleration;
α = the ratio of a to g;
W = the weight of the dam or block.

The horizontal force, P_e, may be assumed acting through the center of gravity of the dam or block—in a downstream direction for a full reservoir, and in an upstream direction for an empty reservoir.

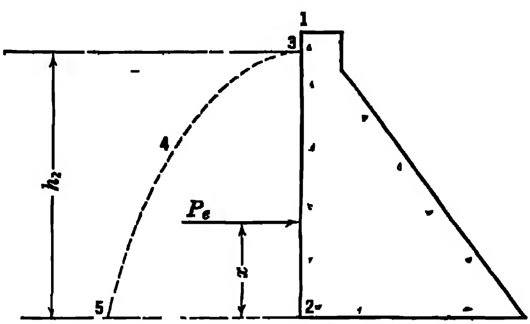


FIG. 8. Earthquake forces on a dam.

(d) *Increased Water Pressure* The inertia of the water in the reservoir also produces a force on the dam, the determination of which is complicated. This problem was admirably treated by Westergaard [8]

The increased water pressure caused by the earthquake on a vertical face can be represented by a diagram of the form 3–4–5–2 in Fig. 8 The true equation of the curve 3–4–5 is complex, but an ellipse or parabola can be assumed without appreciable error A parabola is simpler to use and gives values on the side of safety.

The resulting equation for this earthquake force at any depth, y, is

$$p_e = C_e\alpha h_2 y \qquad [13a]$$

and, for the total force on the dam, is

$$P_e = \tfrac{2}{3}C_e\alpha h_2^2 \qquad [14]$$

where p_e = the additional unit water pressure;
P_e = the additional total water pressure;
α = the ratio of the earthquake acceleration to g;
C_e = a factor depending on physical conditions, principally the height of the dam and the earthquake period;
h_2 = the depth of water on the dam.

This force is located a distance $x = \frac{2}{5}h_2$ above the base of the dam, and its moment about point 2 is

$$P_e x = \tfrac{4}{15}C_e \alpha h_2^3 \qquad [15]$$

For preliminary studies, the value of C_e may be taken as 52 for dams up to 200 ft in height; but, for dams between 200 and 500 ft high, larger values [8] to about 61 should be used.

The authors are not aware of any exact determination of the force caused by the effect of an earthquake on the water for spillway dams during flood. The fact that the water above the crest is not restrained probably reduces somewhat the earthquake force below the crest. It will be conservative to assume that the total force and moment for such cases is the following percentage of the force indicated for non-overflow dams, h_1 and h_2 being as shown in Fig. 3.

$\frac{h_1}{h_2}$	p_f, Percentage of Force	p_m, Percentage of Moment
1.0	0	0
0.8	29	8
0.6	53	26
0.4	74	53
0.2	90	80
0.0	100	100

Equations 14 and 15 consider the masonry face to be essentially vertical and normal to the direction of earthquake motion. This is the condition usually assumed for straight solid gravity dams.

If the upstream face is inclined, as in hollow gravity dams, the most unfavorable direction of the earthquake motion can be considered normal to the inclined face. Under this condition, the pressure on the inclined face can be resolved into horizontal and vertical components as follows:

Horizontal component (same as for vertical face)

$$P_e = \tfrac{2}{3}C_e \alpha h_2^2 \qquad [14]$$

$$P_e x = \tfrac{4}{15}C_e \alpha h_2^3 \qquad [15]$$

Vertical component

$$P_v = \tfrac{2}{3}C_e \alpha h_2^2 \tan\phi \qquad [14a]$$

$$P_v x = \tfrac{2}{3}C_e \alpha h_2^3 \tan^2\phi \qquad [15a]$$

where ϕ = the angle of inclination of the water face of the dam with the vertical.

(*e*) *Inertia of Ice and Silt.* Nothing is known of the effect of earthquake movements on ice and silt pressures. The authors are not aware that the effect of earthquake on ice has ever been considered in the design of dams. As the earthquake period is very small indeed, the presence of the silt would prevent the sudden admission of full headwater pressure and, as mentioned in Section 18, its deleterious effect to any part of the foundation subject to tension. In fact, it is probable that a partial or full vacuum would be induced under such circumstances, increasing the stability of the dam. It is the belief of the authors that the effect would more than balance any increased silt pressure due to earthquake movement, and thus the effect of earthquake on silt deposits may be ignored.

(*f*) *Movements on Faults.* A dam built across a fault on which slippage occurs may be subjected to an immeasurable force, and disruption can be avoided only by providing sufficient flexibility to absorb the motion without damage. Dam foundations crossed by active faults should be avoided. Fault movement is not necessarily confined to the fault on which the earthquake originates, but secondary movements may occur on any active fault in the disturbed area. It is not possible to insure that any prominent fault, although apparently dead, may not be subjected to some movement during an earthquake; however, secondary fractures, bearing no evidence of movement in recent geologic times, involve little danger. Slight movements are not necessarily disastrous.

15. Vertical Reaction of the Foundation. (*a*) *Static Requirements.* In Fig. 9, let ΣW be the resultant of all vertical forces acting on the dam above the foundation and ΣP the resultant of all horizontal forces.* The resultant, R, of ΣW and ΣP will represent then the resultant of all forces.

For the dam to be in static equilibrium, the resultant, R, must be balanced by an equal and opposite reaction of the foundation, consisting of the total vertical reaction equal to ΣW and the total horizontal shear or friction equal to ΣP.

(*b*) *Effects of Elasticity.* Both the dam and the foundation are elastic. The exact effect of such elasticity on the distribution of the foundation stress is not known. A straight-line distribution, as 1–4–3–2 in Fig. 9, is universal practice for dam profiles of the usual form.

(*c*) *Equations for Distribution of Vertical Foundation Reaction.* If the theory of linear distribution is accepted, the unit vertical foundation pressures at any point can be computed from simple rules of mechanics. Let 1–2 of Fig. 9 represent the base of an elemental slice of a loaded dam, the center of

* ΣW and ΣP here represent a general condition and may be applied to either full or empty reservoirs.

gravity of the area of the base being at point 5. Moments of all forces are taken about some arbitrary point of origin 7. Let

W_1, W_2, etc. = the component parts of the total vertical force, ΣW, including uplift but excluding foundation reaction;
P_1, P_2, etc. = the component parts of the total horizontal force, ΣP;
R = the resultant of forces ΣW and ΣP;
x = the lever arm of any force about the origin at 7;
y = the distance of the origin, 7, from the upstream edge of base, 1;
z = the distance from the origin, 7, to the point of intersection, 6, of the resultant, R, with the base;
ΣM_w and ΣM_p = respectively, the summation of moments W_1x, etc. and P_1x, etc. about the origin 7;
e = the eccentricity of loading, or the distance from the center of gravity of the base, 5, to the point of intersection, 6, of the resultant, R, with the base;
m' and m'' = respectively, the distance from the center of gravity of the base to the downstream and upstream ends of the base;
p_r' = the unit vertical foundation pressure at the downstream toe of the base;
p_r'' = the unit vertical foundation pressure at the upstream toe of the base;
A = the area of the base of the dam, 1 2;
I = the moment of inertia of the base about its center of gravity.

Then

$$z = \frac{\Sigma M_w + \Sigma M_p}{\Sigma W} \tag{16}$$

$$e = z - (y + m'') \tag{17}$$

$$p_r' = \frac{\Sigma W}{A} + \frac{(\Sigma W)e}{I} m' \tag{18}$$

$$p_r'' = \frac{\Sigma W}{A} - \frac{(\Sigma W)e}{I} m'' \tag{19}$$

The foregoing equations apply to the case of a full reservoir. For an empty reservoir the resultant, R, would intersect the base to the left of point 5, in Fig. 9, giving a negative value of e and thus altering Eq. 18 and 19 accordingly.

Equations 18 and 19 are general and are independent of the shape of the base of the elemental slice of dam being analyzed. They apply equally to the rectangular base of a foot-thick slice of a straight gravity dam, the tapering base of a slice between radial planes of a curved dam, or the irregular base of the buttress of a hollow dam.

For a foot-thick slice from a straight gravity dam, certain simplifications are possible. For such a slice, the base is a rectangle of 1-ft width and of length

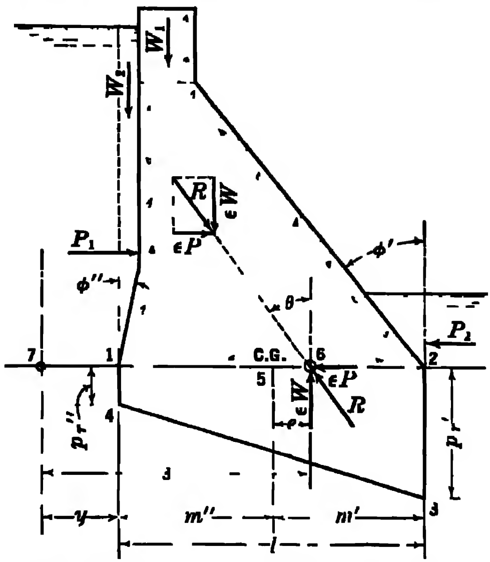


FIG. 9 Elements of foundation-pressure distribution equations

l The center of gravity is at the midpoint, and $m' = m'' = 0.5l$. Also, $I = l^3/12$, and $A = l$. Substituting these values in Eqs 18 and 19 gives

For reservoir full—rectangular base

$$p_r' = \frac{\Sigma W}{l}\left(1 + \frac{6e}{l}\right) \tag{20}$$

$$p_r'' = \frac{\Sigma W}{l}\left(1 - \frac{6e}{l}\right) \tag{21}$$

For reservoir empty—rectangular base

$$p_r' = \frac{\Sigma W}{l}\left(1 - \frac{6e}{l}\right) \tag{20a}$$

$$p_r'' = \frac{\Sigma W}{l}\left(1 + \frac{6e}{l}\right) \tag{21a}$$

In Fig 9, the total vertical foundation reaction, ΣW, for a full reservoir is represented by the trapezoid 1-4-3-2

(*d*) *Uplift and Foundation Reactions Combined* Where uplift exists the total vertical reaction on the base of the dam is assumed to be divided into two parts, as illustrated in Fig 10 The total reaction diagram 1–5–6–2 is divided into uplift 3-4-5-6 and net foundation reaction ΣW, 1-2-3-4 As uplift is included in ΣW and ΣM_u, p_r' (Eq 18) is represented by 2–3 and p_r'' (Eq 19) by 1-4. Total vertical unit reactions can be found thus

$$p_v' = p_r' + p_u' \qquad [22]$$

$$p_v'' = p_r'' + p_u'' \qquad [23]$$

where symbols are as used in Fig. 10. Assuming that the uplift is distributed in accordance with line 7–5 of Fig. 6 as discussed in Section 7, the uplift terms can be computed from Eqs. 7 and 7*a* of that section.

The total vertical unit reactions, p_r' and p_r'', do not correspond to the greater actual, or inclined, stresses, p_i' and p_i'', in the dam and foundation, as indicated in Section 20.

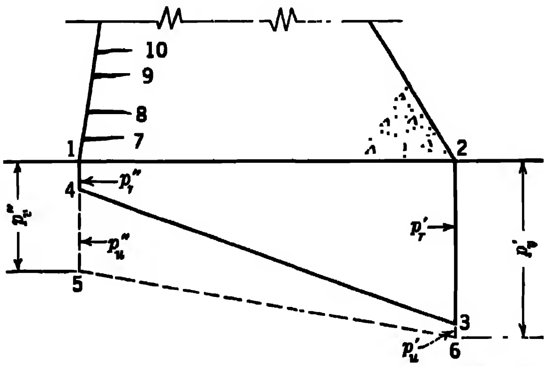


FIG. 10. Combined uplift and foundation pressures.

(*e*) *Requirements for Stability.* It is a general rule that dams of masonry or unreinforced concrete shall be free from tensile stresses. This requires that neither p_r' nor p_r'', Fig. 10, shall be negative.

It is also necessary to limit compressive stresses within the masonry and on the bases of high dams. These stresses are determined from the total vertical unit reactions, p_r' and p_r''. It is probable that the water pressure against fully submerged rock or masonry is not injurious; hence, if uplift could be depended on always to act to its full assumed extent, stresses at the corners of solid gravity dams might be computed from p_r' and p_r''. However, uplift is uncertain. It must be allowed for because it *may* exist, but its existence is never assured. Water under pressure may require years to penetrate to all parts of a carefully constructed dam. The dam must be *safe* either with or without uplift. Consequently, stresses must always be computed on the basis of p_v' and p_v'', values of which can be determined from Eqs. 22 and 23, or they can be computed directly from Eqs. 18 and 19 by omitting uplift from the forces used in the computation.

(*f*) *Law of the Middle Third.* If the value of e in Eq. 20 is greater than $l/6$, p_r'' is negative, i.e., tension. Similarly, p_r' becomes negative for e greater than $l/6$ in Eq. 20*a*.

If tension is not to be permitted, it follows that for a rectangular base the distance from the center of the base to the point at which the line of action of the resultant, R, cuts the base cannot exceed one-sixth of the base length.

This limitation leads to the well-known law of the middle third, which requires that *for a dam slice with a rectangular base* the resultant for all conditions of loading shall fall within the middle third of the base.

This law is merely a means of insuring that neither p_r' nor p_r'' is negative under the usual simple conditions of a solid dam with a rectangular base. If e is numerically equal to exactly one-sixth of l, either p_r' or p_r'' will be zero, depending on the sign of e, and the other will be equal to twice the average pressure.

16. Horizontal Foundation Reactions. Many of the loads on the dam are horizontal or have horizontal components, which must be resisted by frictional or shearing forces along joints in the dam or the foundation. Formerly the general practice was to consider sliding alone, on the theory that the bonding of construction joints is not dependable; the distribution of frictional forces was not considered, then, but only the total force. Inasmuch as no masonry dam founded on ledge rock ever slid on its base until it had sheared through a portion of its foundation or masonry, and usually a substantial portion, it has become common practice to refer to the shear strength or shear resistance of the dam foundation. In gravity dams, allowance for shearing strength can be made in accordance with Eq. 28.

C. Requirements for Stability

17. Causes of Failure. There are two direct ways in which a gravity dam may fail:

1. By sliding (a) on a horizontal or nearly horizontal joint above the foundation, (b) on the foundation, or (c) on a horizontal or nearly horizontal seam in the foundation.
2. By overturning on a horizontal joint (a) within the dam, (b) at the base, or (c) at a plane below the base.

The direct cause of sliding is the presence of horizontal forces greater than the combined shearing resistance of the joint or base and the static friction induced by the vertical forces.

The direct cause of pure overturning, not preceded by some other type of failure, if tension is ignored, is the presence of horizontal forces great enough in comparison with the vertical forces to cause the resultant of all forces acting on the dam above any horizontal plane, including uplift, to pass outside the limits of the dam.

As the resultant approaches the face, the compressive stresses increase rapidly; hence, overturning would be preceded and accelerated by a compression failure. In fact, a dam with the resultant well inside the joint may overturn if the toe of the dam fails by crushing or otherwise so as to reduce the

effective length of the joint or base sufficiently to cause the resultant to pass outside.

A dam may start to overturn but finally fail by sliding. If the resultant passes appreciably outside the middle third (or kern, if the base is of irregular form), a horizontal tension crack may occur, which reduces the shearing strength of the joint or base. Also, the admission of headwater pressure to the fissure increases the uplift, reducing the net reaction and the frictional resistance to horizontal motion. Sliding may result.

In horizontally stratified foundations, sliding failure is more likely to occur along a horizontal joint just below the base than at the base.

18. Rule 1. Location of the Resultant. Section 15 showed that tensile stresses are set up when the resultant falls outside the middle third, for a rectangular base, or when either p_r' or p_r'', as determined by Eq. 18 or 19, is negative. If the joint is incapable of resisting these tensile stresses, the elasticity of the masonry will cause a slight opening of the joint. Such an opening is particularly objectionable at the upstream side when the pond is full, as it may admit full headwater pressure over the entire area not in compression, a condition considerably more severe than usually assumed for uplift. This additional uplift would result in a movement of the resultant toward the toe of the joint, with a further opening of the joint in tension and a further increase in uplift. The progression may be sufficient to cause failure.

Tension at the downstream face can occur only when the reservoir is empty. It is customary to prohibit such tension. The logic of this requirement is open to question, as it is difficult to imagine a dam of the usual type overturning upstream before the water is let into the pond. Tension over as much as 10% of the joint, with the reservoir empty, does not necessarily mean bad design. However, because specifications and codes for the design of dams usually prohibit such tension, the first designing rule can now be written thus:

RULE 1, GOVERNING THE LOCATION OF THE RESULTANT:

Tension shall not exist in any joint of the dam, under any condition of loading. For dams with rectangular joints, this requirement is met if the resultant of all forces, including uplift, acting on the dam above any horizontal joint, for full or empty reservoir, intersects the joint within the middle third. For irregular joints, neither p_r'', reservoir full, nor p_r', reservoir empty (Eqs. 18 and 19), shall be tension. (See also Rule 4 and following exceptions to Rule 1.)

There are two exceptions to this rule. The first is the tacit admission of the existence of tensile strength between the dam and good foundations as indicated in Section 7. The second is the unavoidable presence of tensile stresses at the crest of spillway dams as indicated in Section 37.

19. Rule 2. Resistance to Sliding. (*a*) *Friction Only.* The resultant, ΣP, of all the horizontal forces acting on the dam above any horizontal

joint has a tendency to slide that part of the dam over the lower part. The planes of weakness are the necessary horizontal construction joints, including the joint at the base, and horizontal bedding planes in the foundation below the base. The shearing and frictional resistance must be sufficient to withstand the tendency to slide.

If f' represents the coefficient of static friction of the materials above and below the plane, then $f'\Sigma W$ will be the frictional resistance to sliding.

For equilibrium, neglecting shear $f'\Sigma W$ must be equal to or greater than ΣP. This may be expressed thus:

$$f'\Sigma W = >\Sigma P \tag{24}$$

or

$$\frac{\Sigma P}{\Sigma W} = \tan\theta = < f' \tag{25}$$

where θ is the angle between the vertical and the resultant. This leads to

RULE 2a. RESISTANCE TO SLIDING, SHEAR NEGLECTED:

The tangent of θ, the angle between the vertical and the resultant of all forces, including upilft, acting on the dam above any horizontal plane, shall be less than the allowable coefficient of friction at that plane.

In carefully constructed dams on rock foundations with particular attention paid to obtaining rough surfaces at the base and at construction joints, the coefficient, f', is usually considered to be at least twice as great as indicated by experiments on well-dressed specimens of the same materials. Therefore, if $\tan\theta$ is made equal to or less than the coefficient of friction, as indicated by such tests, a factor of safety in this respect of at least 2 will be provided, and the neglect of the adhesion or shearing resistance at the joints and foundation will serve to increase further the factor of safety. Therefore, for horizontal joints and rock foundations, and neglecting shear, Eq. 25 can be modified for safe design thus:

$$\frac{\Sigma P}{\Sigma W} = \tan\theta = < f \tag{26}$$

where f is the coefficient of friction of the materials on each side of the joint or at the base, as indicated by tests on well-dressed specimens of the same materials.

Values of f for masonry on masonry and masonry on good rock foundations have been assumed variously between 0.6 and 0.75. In general, and for careful work, a value of 0.75 is not excessive. Proper allowance, however, should always be made where the rock foundation is poor, or where it contains nearly horizontal seams close to the finished surface of the foundation. Such seams are particularly dangerous if they contain clay or other unstable material. The allowance to be made will depend on the character of the seam and its contents, its inclination, and the ability of the rock above the seam

to resist movement. Rock otherwise satisfactory may have to be removed in order to eliminate an objectionable seam below it.

A method sometimes used to improve resistance to sliding on horizontally stratified foundations is shown in Fig. 11.

The resultant, R, makes an angle θ with the vertical. If $\tan \theta$ is larger than f, the foundation is excavated to the plane 3–2. The resultant, R', which includes water pressure above point 3 and the weight of concrete 1–2–3, makes a smaller angle, θ, with the normal to the sliding plane 3–2.

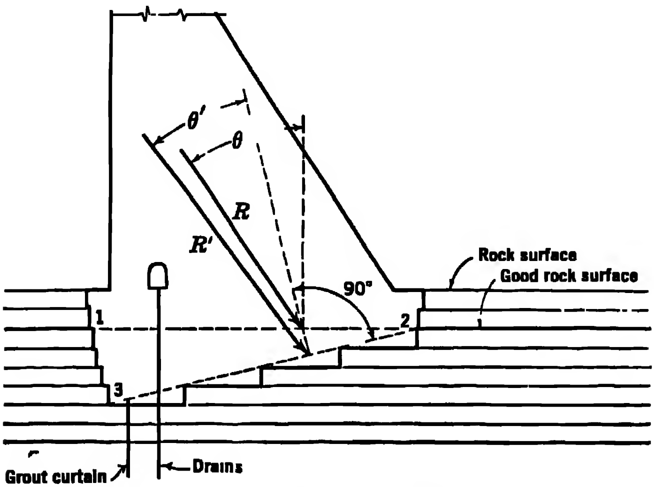


FIG. 11. Precaution against sliding.

Moreover, the allowed coefficient of friction on th which is inclined to the bedding planes, may be greater.

An alternative sometimes used in such cases is to flatten the upstream face of the dam; i.e., increase ϕ'' and x (Fig. 15), thus increasing the vertical component of water pressure to increase the weight of the dam.

On earth foundations, a large factor of safety should be provided because of the danger of sliding on planes below the foundation surface. Artificial bond and increase in the coefficient of friction obviously are ineffective. Equation 26, therefore, should be rewritten for earth foundations thus:

$$\frac{\Sigma P}{\Sigma W} = \tan \theta = \frac{f}{S_f} \qquad [27]$$

where S_f is the factor of safety desired.

For masonry dams on gravel, sand, and clay, approximate values of f are 0.50, 0.40, and 0.30, respectively, but tests on the material should be made. In conservative designs the dam is usually anchored to deep cutoff walls or piles, or a factor of safety of 3 or more is adopted. The weight of the apron,

which is an adjunct of every spillway dam on earth, will assist materially in reducing the tendency to slide.

(b) Combined Shear and Friction. The factor of safety introduced by the neglect of shear under Rule 2a is unknown and variable. Total horizontal and vertical forces vary (at least approximately) as the square of the height of the dam, while shear resistance varies as the first power of the height. Consequently relative safety decreases with increasing height. With increasing heights of structures and improvements in construction operations, it is now permissible to include a definite allowance for shear. This subject was discussed in considerable detail by Henny in 1933.

In considering horizontal joints, the force required to slide the dam, without shear, is added to the force required to shear it, without frictional resistance, and this sum is divided by the total horizontal load to get a factor of safety not lower than a specified minimum. This leads to

RULE 2b, RESISTANCE TO SLIDING, SHEAR INCLUDED:

The total frictional resistance to sliding on any joint, plus the ultimate shearing strength of the joint, must exceed by a safe margin the total horizontal force above the joint for all conditions of loading.

This relationship may be stated algebraically thus:

$$\Sigma P = < \frac{f\Sigma W + r s_a A}{S_{s-f}} \qquad [28]$$

where s_a is the unit shearing strength of the material, S_{s-f} is the shear-friction factor of safety, A is the area of the joint or base, r is the ratio of the average to the maximum shearing stress on the joint, and other symbols are as defined under Eqs. 24, 25, and 26. The friction factor, f, is that for well-dressed specimens.

The value of r can be assumed as 0.5 for preliminary investigations.

The value of s_a can be determined by test if published data for the material involved are not available. It is necessary to know the shear strength of both the foundation and the masonry, the smaller value being used.

Values of s_a from 600 to 1400 lb per sq in. can be used for construction joints in concrete and good rock foundations. However, s_a for poor rock may be much lower, and the dependability of shearing strength, if any, in horizontal bedding planes of some foundations is very problematical.

The value usually adopted for S_{s-f} is 5.

20. Rule 3. Compressive Stresses. Equations for the derivation of the vertical compressive stresses on the base and horizontal joints have been derived in Section 15. The maximum *vertical* compressive stresses are not the maximum stresses that occur in the structure. The maximum stresses are found at the ends of joints, on inclined planes, normal to the face of the dam.

These maximum inclined stresses are approximately determined by the following equations:

At the downstream face,

$$p_i' = p_v' + \tan^2 \phi'(p_v' - p_n') \quad [29]$$

At the upstream face,

$$p_i'' = p_v'' + \tan^2 \phi''(p_v'' - p_n'') \quad [30]$$

where ϕ' and ϕ'' = respectively, the angle with the vertical of the downstream face and the upstream face of the dam;

p_v' and p_v'' = respectively, the total unit vertical reaction at the downstream toe and the upstream toe of the dam, as determined by Eqs. 22 and 23;

p_n' and p_n'' = respectively, the external normal unit pressure due to water and silt on the downstream face and the upstream face of the dam.

RULE 8, GOVERNING COMPRESSIVE STRESSES:

The unit inclined compressive stresses in the dam and the foundation shall not exceed certain prescribed values.

Crushing strengths of concrete up to 4000 lb per sq in. (576,000 lb per sq ft) in 28 days are obtainable, but usual limits for concrete used for gravity dams are from 2000 to 3000 lb per sq in. in 28 days.* The ultimate long-time strengths run higher, depending on the kind of cement used and other conditions. A working stress of one-sixth of the 28-day strength may be considered conservative for gravity dams. Approximately 110,000 lb including earthquake forces, and 78,000 lb excluding earthquake forces, were allowed in the Grand Coulee and Shasta Dams. Allowable stress in buttressed dams is discussed in Chapter 19.

The strength of the foundations on which dams may be founded varies from rock that is much stronger than concrete, through all stages of decomposed and disintegrated rock, which may be very weak, poorly cemented, or lightly consolidated sedimentary rock, down to gravel, silt, and clay deposits. Technical details of tests to determine the strength of rock and soils are beyond the scope of this book.

An entire rock foundation is never unbroken. Fractures are elements of weakness even though consolidated by grouting, the effects of which are difficult to determine. The testing of a rock foundation as a whole is impracticable, and tests on small areas are of no value. Therefore, laboratory tests on samples of the rock must be supplemented by mature judgment in considering the foundation as a whole. Permeability, solubility, and settlement also should receive attention.

The bearing strength of foundation materials is quite variable. Except for low dams on obviously strong rock and very low dams on obviously strong

* Standard test for low-heat cement requires a longer period than 28 days.

sand or gravel, the bearing strength should be carefully investigated by field and laboratory tests. The use of building code or handbook values for foundation strength should be discouraged in important structures.

In a foundation consisting of a single solid rock mass, free from any kind of jointing that could relieve lateral pressures caused by lateral expansion under vertical load, the bearing strength is no doubt considerably in excess of the laboratory strength, and it may be permissible under favorable circumstances to permit the working stress to approach the laboratory breaking strength. This should be done, however, only under the advice of experienced experts.

For a foundation of good, compact, but jointed rock, the conservative designer usually will consider that the ultimate strength of the rock "en masse" is not appreciably greater than the breaking strength of laboratory samples, and will apply a safety factor of 4 to 7 in selecting an allowable working stress. Specimens selected for testing should be representative and should not consist entirely of either the best or the poorest materials. Allowance must be made for the fact that joint planes must be excluded from laboratory samples.

Whether the maximum pressure in the foundation is equal to the inclined pressure in the dam at the base or to the vertical pressure only is frequently the subject of debate. Undoubtedly, stress conditions change rapidly in the rock beneath the toe, but it seems rational and on the side of safety to assume that the maximum stress in the rock in immediate contact with the base of the dam equals the inclined toe stress in the dam.

It is sometimes necessary to build masonry dams on foundations other than rock. The bearing and shearing strengths of such materials are variable. Usually the dams are low, and provision against underflow and sliding results in a width of base ample for bearing strength. Building codes may be used as a rough guide to bearing values in such cases. The New York Building Code allows the following for buildings:

	Lb per Sq Ft
Gravel	12,000
Coarse sand	8,000
Firm clay	4,000
Soft clay	2,000

If an important masonry structure involving large forces, or any structure impounding much water, is founded on material other than rock, a careful investigation should be made by an expert in soil machanics. It is not possible to write a simple specification for such an investigation. Foundation materials having a dry weight of less than 100 lb per cu ft should be looked on with grave suspicion. Such loose foundation materials generally should be removed.

Piling ordinarily is undesirable for the support of dams on soft foundations and should be avoided or used with extreme care. The weight of the structure

being supported on the piles, the foundation materials may remain, become soft and porous, or settle entirely away from contact with the base, with consequent danger of underflow or piping. Dams founded on piles should be provided with ample cutoffs, upstream aprons, drains, or other suitable means of controlling percolation and piping. Means for accomplishing these purposes are discussed in Chapter 25.

21. Rule 4. Tension on Interior Planes. Tension in any horizontal plane is undesirable, as indicated in Section 18. As a matter of fact, it is common practice with two exceptions, which are indicated in Section 18, to prohibit tension in any part of the structure.

RULE 4, GOVERNING INTERNAL TENSION:
The dam shall be designed and constructed in such a manner as to avoid or provide adequately for tension on horizontal, inclined, or vertical interior planes. (See exceptions previously noted.)

Positive values of p_r' or p_r'', as determined by Eqs. 18 and 19, insure no tension in horizontal planes. For dams tentatively designed by the "single-step" method subsequently advocated for preliminary studies, and with the usual assumptions of uplift, tension in inclined and vertical planes may be assumed as nonexistent. Possible tension in interior planes, around voids, etc., is subject to local treatment.

22. Rule 5. Margin of Safety. All design factors contributing to the permanent safety of a dam should be chosen with care and should be conservative. A careful estimate of the weight of the dam should not vary more than 1 or 2% from the actual weight, and the weight and pressure of the water are closely known. The maximum depth of water should include liberal allowance for the highest possible flood, and waves and seiches should be provided for if required, in order that the assumed water load surely shall not be exceeded. Allowances for uplift, earthquake forces, silt, and ice pressures must be adequate. The assumed safe sliding factor, foundation strength, and concrete or masonry strength must be conservative. If all these factors are carefully chosen, the dam, if properly designed and constructed, will be safe. If the foundation is rock, there is an additional element of safety because of the adhesion of the concrete to the foundation. To this feature alone can be attributed the continued existence of a number of poorly designed dams. These considerations lead to

RULE 5, GOVERNING THE MARGIN OF SAFETY:
All assumptions of forces acting on the dam shall be unquestionably on the safe side; all unit stresses adopted in design shall provide an ample margin against rupture; and the adopted safe sliding factor or shear-friction safety factor shall be conservative.

The term "factor of safety" as used in structural design is directly applicable to hollow dams and arch dams through the stress equations, but the term is less directly applicable to gravity dams. It is sometimes said that a

gravity dam, with the resultant, reservoir full, at the downstream middle third point has a factor of safety of 2 against overturning. This statement comes from the fact that doubling the active overturning moment in such a dam of triangular form with vertical upstream face will move the resultant out to the downstream face. Actually, failure by crushing would occur before the resultant reached the face. On the other hand, the possibility of doubling the moment is nonexistent; hence, a numerical factor of safety against overturning is meaningless. The same is true of sliding. The shear-friction safety factor and the safety factor against a compression failure have the ordinary structural significance.

23. Rule 6. Design and Construction. When the shape of the section of the dam has been determined in accordance with established rules, careful attention must be given to the details of the design and the methods of construction, so that the structure may be satisfactory in every respect.

RULE 6, GOVERNING DETAILS OF DESIGN AND METHODS OF CONSTRUCTION:

All details shall support and conform to the assumptions used in the design; the masonry shall be of a quality suited to the working stresses adopted and shall be practically watertight and durable; protection against overflowing water shall be ample.

D. General Procedure of Design

24. General Considerations. A gravity dam must conform at all elevations to each of the rules established in Sections 18 to 23 The relative influence of these rules on the design varies with height. Near the top all stability requirements are met if a reasonable top thickness is provided; next, compliance with Rule 1 assures compliance with all other rules; and the other rules come into ascendency, in turn. The designer usually knows, at least approximately, which rules govern at a given elevation and may prepare his design according to such rules, later checking for compliance with others and redesigning where necessary.

Each of the design rules is simple and, within limits, may be expressed algebraically. However, because of the many variables involved and the necessity of changing from one governing rule to another at different heights, it is not practicable to write a set of equations from which all the dimensions of a gravity dam can be directly determined. The only practicable solution is to design the dam, joint by joint, beginning at the top, making each joint conform to all rules.

This procedure results in a dam with polygonal faces that may be smoothed up for appearance with no appreciable change in stability or economy. This is known as the "multiple-step design."

A diagrammatic sketch showing the zones of the multiple-step design is given in Fig. 12. The zones can be briefly described as follows:

For Non-overflow Dams (Fig. 12a)

Zone I. Practical considerations—must resist ice pressure.

Zone II. Both faces vertical—at bottom of zone, resultant for reservoir full reaches extremity of middle third.

Zone III. Downstream face begins to batter to keep resultant for reservoir full at extremity of middle third at bottom of zone; resultant, reservoir empty, reaches extremity of middle third.

Zone IV. Both faces battered to keep resultants, reservoir full and empty, at extremity of middle third—at bottom of zone, stresses for reservoir full reach allowed values.

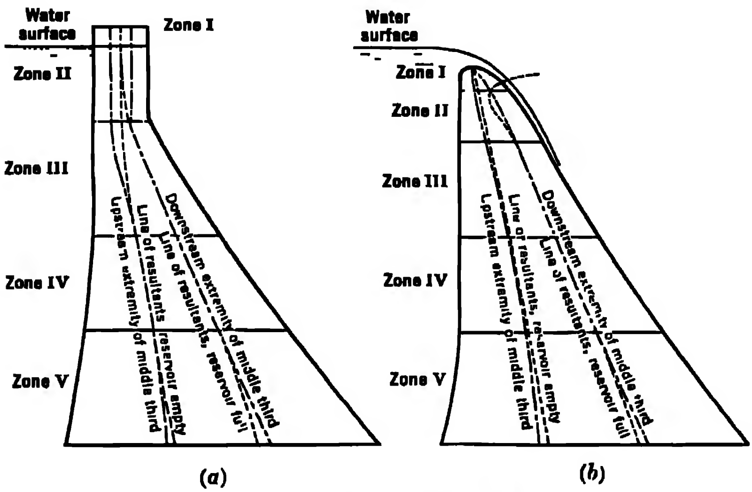


FIG. 12. Zoning for dams.

Zone V. The determination of the section below the bottom of Zone IV is very complicated for the multiple-step design,* and, moreover, is not applicable to the single-step design advocated subsequently for preliminary designs.

For Spillway Dams (Fig. 12b)

Zone I. Crest shaped to fit jet of spilling water. (See Section 33.) Resultant for reservoir full is outside middle third. Tension exists. (See Section 36.)

Zone II. Downstream face shaped to fit jet of spilling water. At bottom of zone, resultant for reservoir full reaches extremity of middle third.

Zone III. Downstream face no longer fits jet but must have a flatter slope to keep resultant for reservoir full at extremity of middle third. For spillway dams, the resultant, reservoir empty, seldom if ever reaches the extremity of the middle third, and the possibility of the resultant's being outside the

* See p. 342 of Ref. 1, Section 57.

middle third for reservoir empty can be neglected, at least for preliminary designs.

Zone IV. At the bottom of this zone the stresses for reservoir full reach allowed values.

Zone V. Same as explained for non-overflow dams.

The multiple-step method is always used for the final design of dams within a height that does not encroach greatly on Zone V. Examples of multiple-step designs are shown in Figs. 13, 14, and 25.

The single-step method is always used for final designs of very high dams that extend well beyond Zone V. Examples of single-step designs are shown in Figs. 13, 14, 16, 17, and 25.

The theory of the multiple-step design is fully described in current literature,* to which reference should be made for final designs where applicable, as indicated previously. The single-step method not only is directly applicable to very high dams, but also can be used within an accuracy of 2 to 4%, on the safe side, for preliminary designs to obtain the area of the maximum section of the dam.

For non-overflow dams with a ratio of top width to height of dam of 0.75 or more, the error is zero. For a ratio of about 0.10, the error is about 2 or 3%, gradually reducing to approximately zero for a ratio of zero.

For spillway dams with a ratio of head on crest to height of dam of 0.5 or more, the error is zero. For ratio of about 0.10, the error is about 3 to 4%, gradually reducing to around zero for a ratio of zero.

E. Solid Gravity Non-overflow Dams

25. Multiple-step Design. As mentioned before, the multiple-step method should be used for all final designs for dams of a height within which the multiple-step method is applicable, although the single-step method is recommended for preliminary studies.

Figures 13 and 14 show sections of non-overflow dams obtained by the multiple-step method under the following assumptions. For comparison, single-step design sections are superimposed.

Example 1

Maximum depth of water	200	ft
Depth of tailwater	0	
Top width	24	ft
Weight of concrete	150	lb per cu ft
Weight of water	62.5	lb per cu ft
Uplift intensity factor, ζ	0.5	
Allowable coefficient of friction, f	0.75	

Allowable compression does not govern for this height.

No earthquake.

* See Ref. 1. Section 57.

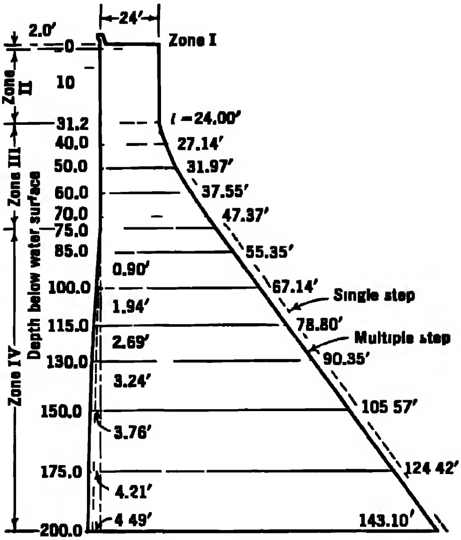


Fig. 13. Example 1.

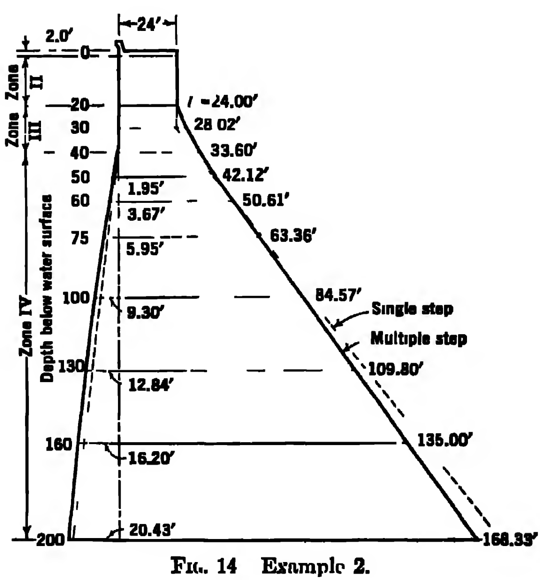


Fig. 14 Example 2.

Example 2. Same as Example 1 but with earthquake acceleration of an intensity of $0.1g$.

Figure 25 also shows a comparison of the single- and multiple-step designs for a spillway dam.

26. Single-step Design. A diagrammatic sketch for the single-step design is given in Fig. 15. The downstream face is a straight line that, when ex-

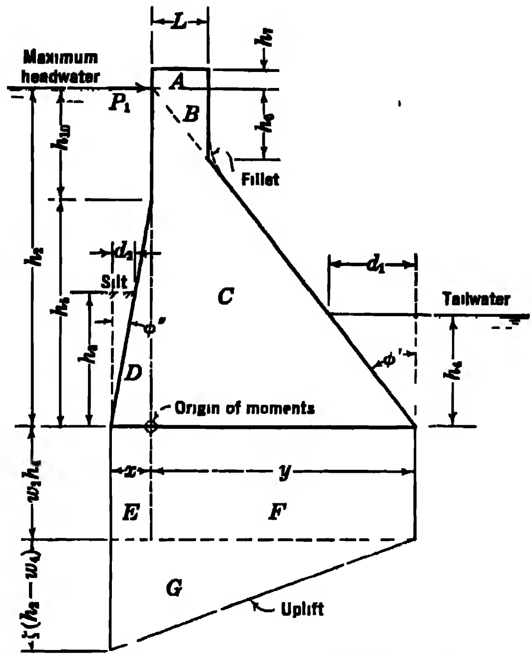


FIG. 15. Diagrammatic sketch explaining single-step design for non-overflow dams

tended, intersects the upstream face at headwater surface. The width, L, of the top of the dam must be wide enough for practical considerations, which vary for each case. Without earthquake the most economical top width is 10 to 15% of the height [9]. With earthquake assumptions a heavy top section is disadvantageous. A superelevation of the top above high water is usually desirable to get beyond the reach of waves.

Where considerable depth over an ungated spillway is required to pass maximum floods, high headwater and ice pressure will not occur simultaneously. The section should be checked for the condition of maximum headwater and no ice pressure and also for the condition of normal high headwater with ice pressure. For gated spillways, normal high headwater may be the same as flood level, the gates being opened enough to pass excess flow

TABLE 1

BASIC EQUATIONS FOR SOLID NON-OVERFLOW DAMS

(SINGLE-STEP DESIGN)

Line	*Item*	*Force*	*Lever*
	VERTICAL FORCES		
1	Concrete—Rectangle *A*	$+w_1Lh_7$	$+\frac{L}{2}$
2	" Triangle *B*	$+\frac{w_1Lh_6}{2}$	$+\frac{2L}{3}$
3	" Triangle *C*	$+\frac{w_1h_2y}{2}$	$+\frac{y}{3}$
4	" —Triangle *D*	$+\frac{w_1h_5x}{2}$	$-\frac{x}{3}$
5	Headwater	$+w_2h_{10}x$	$-\frac{x}{2}$
6		$+\frac{w_2h_5x}{2}$	$-\frac{2x}{3}$
7	Tailwater	$+\frac{w_2h_4d_1}{2}$	$+y-\frac{d_1}{3}$
8	Silt *	$+\frac{w_4h_3^2x}{2h_5}$	$-x\left(1-\frac{h_3}{3h_5}\right)$
9	Uplift - Rectangle *E*	$-w_2h_4x$	$-\frac{x}{2}$
10	" —Rectangle *F*	$-w_2h_4y$	$+\frac{y}{2}$
11	" —Triangle *G*	$-w_2\zeta(h_2-h_4)\left(\frac{x}{2}+\frac{y}{2}\right)$	$+\frac{y}{3}-\frac{2x}{3}$
12		ΣW	
	HORIZONTAL FORCES		
13	Headwater	$+\frac{w_2h_2^2}{2}$	$+\frac{h_2}{3}$
14	Ice	$+P_i$	$+h_2$
15	Tailwater	$-\frac{w_2h_4^2}{2}$	$+\frac{h_4}{3}$
16	Silt *	$+\frac{w_4h_3^2\sigma}{2}$	$+\frac{h_3}{3}$
	EARTHQUAKE		
17	Concrete—Rectangle *A* †	$+\alpha$ (Line 1)	$+h_2+\frac{h_7}{2}$

TABLE 1 (*Continued*)

BASIC EQUATIONS FOR SOLID NON-OVERFLOW DAMS

(SINGLE-STEP DESIGN)

Line	*Item*		*Force*	*Lever*
18	"	-Triangle B †	$+\alpha$ (Line 2)	$+h_2 - \frac{h_8}{3}$
19	"	Triangle C †	$+\alpha$ (Line 3)	$+\frac{h_2}{3}$
20	"	Triangle D †	$+\alpha$ (Line 4)	$+\frac{h_5}{3}$
21	Water		$+\frac{2}{3}C_e\alpha h_2^2$	$+0.4h_2$
22	Ice		Ignored	
23	Silt		Ignored	
24			ΣP	

* $\sigma = \frac{1 - \sin\phi}{1 + \sin\phi}$ from Eq. 8. Lines 8 and 16 are for reservoir full. Use saturated silt w_5 for reservoir empty.

† Use negative force for reservoir empty.

and having a capacity equal to the maximum flood. In this case, the ice pressure may occur with maximum headwater.

The contingency that earthquake and maximum flood will occur simultaneously is considered too remote. Therefore, the dam should be tested for maximum headwater level and no earthquake and also for normal high headwater and earthquake.

The upstream face should be kept vertical for a distance, h_{10}, equal to about $3L$ for the condition of no earthquake, and equal to about $2L$ for the condition of earthquake.

Without appreciable error, the values of h_6 and d_1 can be assumed equal to $1.33L$ and $0.75\ h_4$, respectively. High tailwater is sometimes difficult to determine. The depth of tailwater may be advantageous to some rules of design and may have the reverse effect on others. Therefore, the dam should be tested for the maximum possible and minimum possible tailwater depth at the time of maximum headwater.

After the design is completed, the section above the bottom of the vertical downstream face, i.e., for depth h_6 below headwater, should be tested for stability, particularly if an earthquake assumption is used. A fillet, as indicated in Fig. 15, may be necessary. However, the added volume is negligible. It will be on the safe side to adopt the upper part of the computed maximum

section for the area of those parts of the dam that are less than the maximum height. That is, the area of the upper half of the computed maximum section may be about 8% larger than required.

If the section across the site is triangular, the error in adopting the computed shape of the maximum section throughout may run about 4 or 5% on the safe side. If, however, a considerable portion of the dam is constant in height and somewhat less than the maximum section, it would be advisable to compute another section for that part.

27. Basic Equations for Single-step Design. The basic equations given in Table 1 will be found useful in the design of solid non-overflow dams by the single-step method. Nomenclature is given in Fig. 15 and in Section 3

Moments are taken about the base of the section, on a vertical extension of the vertical top part of the upstream face. Clockwise moments are positive, as are downward vertical forces and horizontal forces directed to the right. Moments have been omitted from the table because it is more convenient to derive the forces and levers from the equations given and multiply them to get the moments than to use equations for moments.

The following examples will indicate the application of the foregoing equations to the design of a dam by the single-step method.

28. Example 3. 200-ft Non-overflow Solid Gravity Dam. Let it be required to estimate the approximate cross-section of a non-overflow gravity dam 200 ft high on a good foundation. Assume that the spillway crest is gated so that normal high-water elevation and maximum high-water elevation during floods are equal. In this case, highest headwater, ice, and earthquake can be assumed to occur simultaneously. Let

H = total height of dam	= 205 ft
h_2 = depth of headwater	= 200 ft
h_3 = depth of silt	= 50 ft
h_4 = depth of tailwater	= 30 ft
$h_{10} = 2L = 2 \times 24$	= 48 ft
$h_5 = h_2 - h_{10}$	= 152 ft
$h_6 = 1.33L$	= 36 ft
h_7 = 5 ft	
$d_1 = 0.75h_4$	= 22.5 ft
$d_2 = xh_3/h_5$	$= x/3.04$
L = width of top	= 24 ft
w_1 = weight of concrete	= 150 lb per cu ft
w_2 = weight of water	= 62.5 lb per cu ft
w_3 = dry weight of silt	= 100 lb per cu ft
k = percentage of voids in silt, expressed as a decimal	= 0.40
w_4 = submerged weight of silt	= 63 lb per cu ft
w_5 = saturated weight of silt	= 125 lb per cu ft
ϕ = angle of internal friction for silt	= 30°
$\sigma = \dfrac{1 - \sin\phi}{1 + \sin\phi}$	$= \dfrac{1}{3}$

ζ = uplift intensity factor = 0 5
P_i = ice pressure = 10,000 lb per lin ft of dam
f = allowable coefficient of friction = 0 75
r = ratio of maximum to average shear = 0 5
s_a = ultimate shear resistance of the foundation = 800 lb per sq in or, as an alternative, zero
$S_{s\text{-}f}$ = minimum permissible shear-friction safety factor = 5 0 (investigation for shear required only when tan θ is greater than f = 0 75)
p_i = maximum allowable inclined stress in dam or foundation = 50,000 lb per sq ft
α = ratio of earthquake acceleration to gravity = 0 1
C_e = earthquake factor = 51 7

TABLE 2

Computations for Example 3

Line	Item	Force	Lever	Moment
	VERTICAL FORCES			
1	Concrete Rectangle A	$+150 \times 24 \times 5$ [illegible] $+18\,000$	+ [illegible]	$+216\,000$
2	Triangle B	$+150 \times 24 \times$ [illegible] $= +64\,800$	$+2 \times$ [illegible]	$+1\,036\,000$
3	− Triangle C	$+150 \times 200y/2 = +15\,000y$	$+y/3$	$+5000y^2$
4	Triangle D	$+150 \times 152x/2 = +11\,400x$	$x/3$	$-3800x^2$
5	Headwater	$+62.5 \times 48x = +3000x$	$-x/2$	$-1500x^2$
6		$+62.5 \times 152x/2 = +4750x$	$-x/3$	$-3170x^2$
7	Tailwater	$+62.5 \times 30 \times 22.5/2 = +21\,080$	$+(y - 22.5/3)$	$+21\,080y$ $-158\,000$
8	Silt			
8A	Reservoir full	$+63 \times 50^2x/2 \times 152 = +518x$	$-x\left(1 - \frac{50}{3 \times 152}\right) = -0.89x$	$-461x^2$
8B	Reservoir empty	$+125 \times 50^2x/2 \times 152 = +1027x$	$-x\left(1 - \frac{50}{3 \times 152}\right) = -0.89x$	$-914x^2$
9	Uplift Rectangle E	$-62.5 \times 30x = -1875x$	$-x/2$	$+937x^2$
10	−Rectangle F	$-62.5 \times 30y = -1875y$	$+y/2$	$-937y^2$
11	−Triangle G	$-62.5 \times 0.5(200 - 30)x/2 = -2656x$	$+y/3$	$-885xy$
11A			$2x/3$	$+1770x^2$
11B		$-62.5 \times 0.5(200 - 30)y/2 = -2656y$	$+y/3$	$-885y^2$
11C			$-2x/3$	$+1770xy$
12		ΣW		
	HORIZONTAL FORCES			
13	Headwater	$+62.5 \times 200^2/2 = +1\,250\,000$	+ [illegible]	$+83\,333\,000$
14	Ice	$+10\,000$	$+200$	$+2\,000\,000$
15	Tailwater	$-62.5 \times 30^2/2 = -28\,120$	+ [illegible]	$-281\,200$
16	Silt			
16A	Reservoir full	$+63 \times 50^2/2 \times 3 = +26\,200$	+ [illegible]	$+437\,000$
16B	Reservoir empty	$+125 \times 50^2/2 \times 3 = +52\,100$	+ [illegible]	$+867\,000$
	EARTHQUAKE			
17	Concrete* Rectangle A	$+0.1 \times 18\,000 = +1800$	$+200 +$ [illegible]	$+364\,700$
18	Triangle B	$+0.1 \times 64\,800 = +6480$	$+200 -$ [illegible]	$+1\,217\,000$
19	* Triangle C	$+0.1 \times 15\,000y = +1500y$	+ [illegible]	$+100\,000y$
20	Triangle D	$+0.1 \times 11\,400x = +1140x$	+ [illegible]	$+57\,800x$
21	Water	$+\frac{2}{3} \times 51.7 \times 0.1 \times 200^2 = +138\,000$	$+0.4 \times 200$	$+11\,040\,000$
24		ΣP		ΣM

* For reservoir empty reverse the sign of the forces and moments

For Reservoir Empty

$$\Sigma W_E = +12{,}427 \times 15{,}000y + 82{,}800 \quad [31]$$

$$\Sigma P_E = -1140x - 1500y + 43{,}720 \quad [32]$$

$$\Sigma M_E = -4714x^2 - 57{,}800x + 5000y^2 - 100{,}000y + 537{,}300 \quad [33]$$

For Reservoir Full

$$\Sigma W_F = +15{,}137x + 10{,}469y + 103{,}880 \quad [34]$$

$$\Sigma P_F = +1140x + 1500y + 1{,}404{,}360 \quad [35]$$

$$\Sigma M_F = -6270x^2 + 57{,}800x + 885xy + 3178y^2 + 121{,}080y + 99{,}204{,}500 \quad [36]$$

The following equations can be used to obtain the values of x and y in order to fulfill the requirements of Rule 1 regarding the location of the resultant for the reservoir both empty and full.

Reservoir Empty

$$\Sigma W_E \left(\frac{y}{3} - \frac{2x}{3}\right) = \Sigma M_E \quad [37]$$

Reservoir Full

$$\Sigma W_F \left(\frac{2y}{3} - \frac{x}{3}\right) = \Sigma M_F \quad [38]$$

The values derived in Eqs. 31 to 36 are substituted in Eqs. 37 and 38, resulting in two simultaneous equations in x and y.

A value of y equal to $0.75h_2$ is assumed for the first trial and the resulting value of x derived to fulfill the requirements of Rule 1 with the reservoir empty. Then, this value of x being assumed correct, it can be substituted to find a more correct value of y to fulfill the requirements. This process is continued until an agreement is reached. Actually, two or three trials will give accurate results of Rule 1 for reservoir full.

Proceeding in this manner, we will find that, for this example, the first-trial values of x and y are 19.6 and 154.5, respectively. A second trial would result in values of x and y equal to 19.7 and 154.0, respectively. This difference would result in a variance in estimated quantities that is insignificant, considering the usual uncertainty in the estimated level of a satisfactory foundation.

With the second-trial values of x and y, the final forces and moments are

$$\Sigma W_F = +2{,}013{,}540 \text{ lb}$$

$$\Sigma W_E = +2{,}638{,}000 \text{ lb}$$

$$\Sigma P_F = +1{,}658{,}000 \text{ lb}$$

$$\Sigma P_E = -210{,}000 \text{ lb}$$

$$\Sigma M_F = +194{,}659{,}000 \text{ ft-lb}$$

$$\Sigma M_E = +100{,}770{,}000 \text{ ft-lb}$$

The resulting section is shown in Fig. 16.

To test for resistance to sliding, if no shearing stress is allowed, i.e., to comply with Rule 2a when $s_a = 0$, we find, from Eq. 26, that

$$\frac{\Sigma P_r}{\Sigma W'_p} = \tan \theta = \frac{1,658,000}{2,013,540} = 0.824$$

Since this is in excess of the allowed value of $f = 0.75$, the expedients described in Section 19 should be resorted to, if the foundation has no shearing resistance.

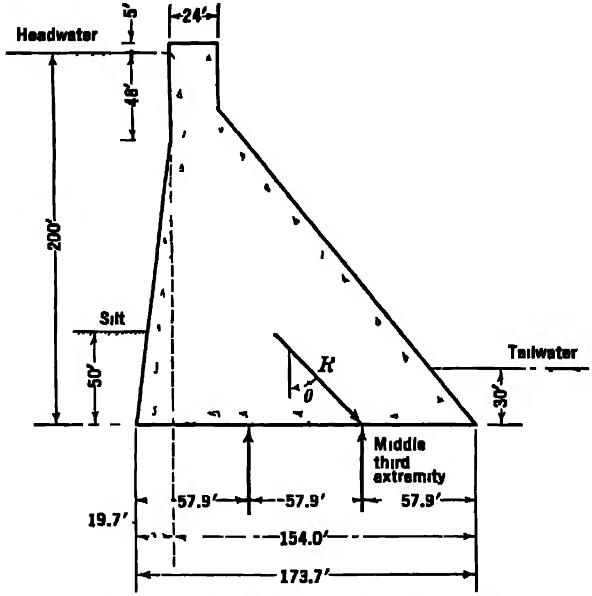


FIG. 16. Elements of the section for Example 3.

To test for Rule 2b, for sliding with the foundation capable of a shearing resistance of $s_a = 800$ lb per sq in. or 115,200 lb per sq ft, we find, from Eq. 28, that

$$S_{s\text{-}f} = \frac{0.75\Sigma W'_p + rs_aA}{\Sigma P_r}$$

$$= \frac{0.75 \times 2,013,540 + 0.5 \times 115,200 \times 173.7}{1,658,000} = 9.12$$

This factor of safety, considering friction and shear, is in excess of the allowed value of $S_{s\text{-}f} = 5.0$, and the condition is satisfactory.

To test for Rule 3, governing compressive stresses, the stresses are determined according to Section 20 as follows:

The vertical foundation reactions can be found from Eqs. 20, 21, 20*a*, and 21*a*, in which, for this case where the resultants hit the base at the extremity of the middle third, $e = l/6 = 173.7/6 = 28.95$.

Reservoir Full

Downstream toe

$$p_n' = \frac{2{,}013{,}540}{173.7}\left(1 + \frac{6 \times 28.95}{173.7}\right) = 23{,}160 \text{ lb per sq ft}$$

Upstream toe

$$p_r'' = \frac{2{,}013{,}540}{173.7}\left(1 - \frac{6 \times 28.95}{173.7}\right) = 0$$

Reservoir Empty

Downstream toe

$$p_r' = \frac{2{,}638{,}000}{173.7}\left(1 - \frac{6 \times 28.95}{173.7}\right) = 0$$

Upstream toe

$$p_r'' = \frac{2{,}638{,}000}{173.7}\left(1 + \frac{6 \times 28.95}{173.7}\right) = 30{,}340 \text{ lb per sq ft}$$

The uplift pressures can be found from Eqs. 7 and 7*a* as follows:

Reservoir Full

Downstream toe

$$p_u' = 62.5 \times 30 = 1875 \text{ lb per sq ft}$$

Upstream toe

$$p_u'' = 62.5[30 + 0.5(200 - 30)] = 7150 \text{ lb per sq ft}$$

Reservoir Empty

Upstream and downstream toes = 0

The total vertical reactions, including uplift, obtained from Eqs. 22 and 23 are

Reservoir Full

Downstream toe

$$p_v' = 23{,}160 + 1875 = 25{,}035 \text{ lb per sq ft}$$

Upstream toe

$$p_v'' = 0 + 7150 = 7150 \text{ lb per sq ft}$$

Reservoir Empty

Downstream toe

$$p_v' = 0 + 0 = 0 \text{ lb per sq ft}$$

Upstream toe

$$p_v'' = 30{,}340 + 0 = 30{,}340 \text{ lb per sq ft}$$

The inclination of the faces of the dam are

Downstream

$$\tan \phi' = \frac{y}{h_2} = \frac{154}{200} = 0.770$$

Upstream

$$\tan \phi'' = \frac{x}{h_5} = \frac{19.7}{152} = 0.130$$

The normal pressures on the faces of the dam are easily found to be

Reservoir Full

Downstream toe

$$p_n' = 1875 \text{ lb per sq ft}$$

Upstream toe

Silt	1,440
Water	13,500

$$p_n'' = 14{,}940 \text{ lb per sq ft}$$

Reservoir Empty

Downstream toe

$$p_n' = 0$$

Upstream toe

Silt	2860
Water	0

$$p_n'' = 2860 \text{ lb per sq ft}$$

The maximum inclined stresses, from Eqs. 29 and 30, are:

Reservoir Full

Downstream toe

$$p_i' = 25{,}035 + 0.77^2(25{,}035 - 1875) = 38{,}760 \text{ lb per sq ft}$$

Upstream toe

$$p_i'' = 7150 + 0.13^2(7150 - 14{,}940) = 7018 \text{ lb per sq ft}$$

Reservoir Empty

Downstream toe

$$p_i' = 0 + 0.77^2(0 - 0) = 0$$

Upstream toe

$$p_i'' = 30{,}340 + 0.13^2(30{,}340 - 2860) = 30{,}800 \text{ lb per sq ft}$$

It is seen that the maximum inclined stress is 38,760 lb per sq ft, which is well below the allowed 50,000 lb per sq in.

Should the computed stress exceed the allowed limit, then the base of the dam is widened. Solution is accomplished by trial. An arbitrarily assumed base length, $x + y$ (Fig. 15), can be placed in successive trial positions until the position is found that, without violating Rules 1 or 2, gives (if possible) satisfactory values for both p_i', reservoir full, and p_i'', reservoir empty. If this cannot be accomplished with a given trial length, a longer length is assumed and the trial is repeated. If there is strength to spare, a shorter length can be tried, the process being continued until a base length is found that at its best position gives the allowable inclined stresses at the two faces simultaneously.

For preliminary estimates, it is not necessary to test the dam for Rule 4, for tension in inclined planes. (See Section 21.)

29. Example 4. 350-ft Non-overflow Solid Gravity Dam. Figure 17 shows a section of a 350-ft non-overflow dam on foundations such that limit-

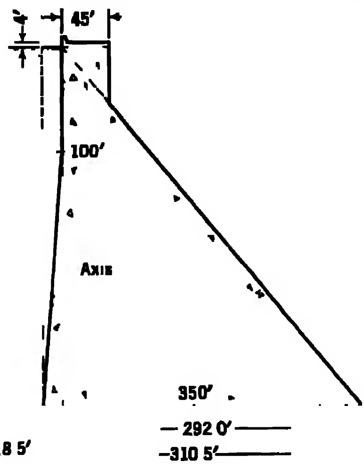


FIG. 17. Single-step design. Example 4.

ing pressures govern the design. For this case the single step method would be used for the final design. The design data are as follows:

h_2	= maximum depth of headwater	= 350 ft
h_4	= depth of tailwater	= 0
L	= top width	= 45 ft
w_1	= weight of concrete	= 150 lb per cu ft
w_2	= weight of water	= 62.5 lb per cu ft
ζ	= uplift intensity factor	0.5
f	= allowable coefficient of friction	= 0.7
s_a	= ultimate shearing strength of the foundation and dam	= 800 lb per sq in
S_f	= minimum permissible shear friction safety factor	= 5.0
p_i	= maximum allowable inclined stress on dam or foundation	= 50,000 lb per sq ft

No earthquake.

30. Notable Non-overflow Solid Gravity Dams. Figures 18a and 18b show examples of notable non-overflow dams in this country. It must be understood that special conditions such as the strength of the foundation, earthquake assumptions, the ideas of the designer, etc., greatly affected the shape of the sections.

31. Approximate Quantities in Non-overflow Solid Gravity Dams. Figure 19 shows approximate quantities of concrete in non-overflow dams the foundations of which are such that the allowed tan θ (Fig. 9) is at least 0.76 and the strength of the foundation is sufficient to withstand the stresses. An uplift intensity factor of 0.5 is assumed. No silt, ice, tailwater, or earthquake is assumed. The weight of concrete is 140 lb per cu ft.

The use of this diagram where concrete actually weighs 150 lb per cu ft will give a degree of conservatism of about 4%.

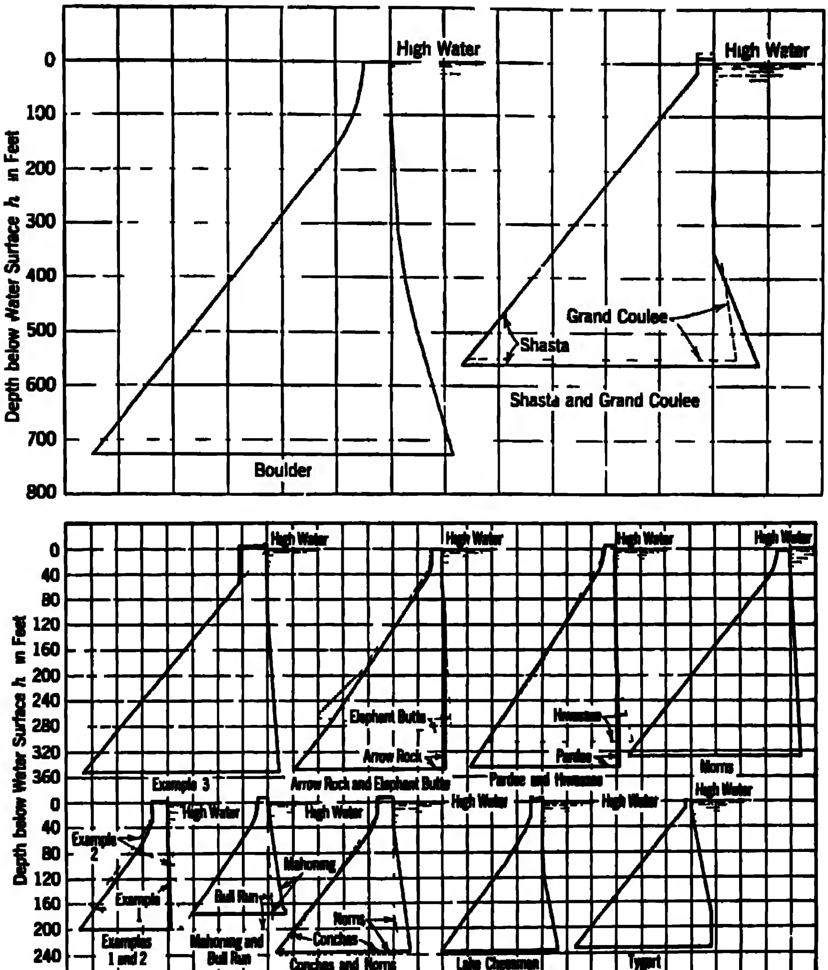


FIG. 18a. Profiles of notable non-overflow gravity dams.

B. SOLID GRAVITY SPILLWAY DAMS

32. Methods of Design. The general method of determining the stability of solid spillway dams differs in no way from that previously described for solid non-overflow dams except in the vicinity of the crest where, as previously mentioned, the section should be proportioned to fit the lower nappe of the sheet of water spilling over the dam during maximum flood. The preceding principles of design will be supplemented as required but will not be repeated here.

33. The Shape of the Crest. If the sheet of water spilling over the dam leaves the face of the dam, there is danger of the formation of a partial vacuum under the sheet with a resultant additional overturning force on the dam. Therefore, except for special conditions mentioned subsequently, it is desirable to shape the crest and the downstream face to completely fill the

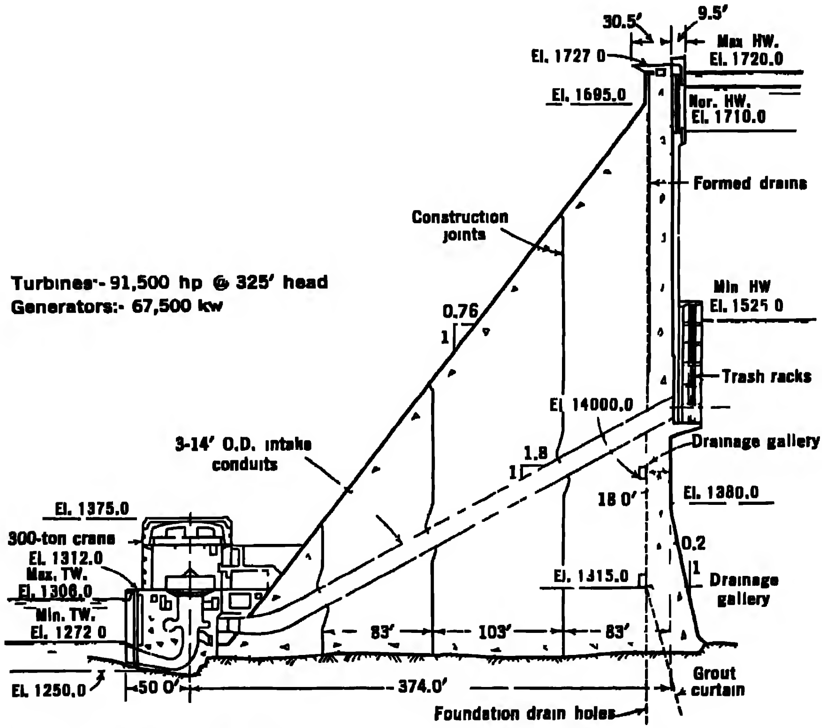


Fig. 186 Section through Fontana Dam and powerhouse (*Eng News-Rec*, Nov 16, 1944)

space under a freely discharging jet corresponding to the maximum flood expected

Experiments have been made to determine the shape of the sheet of water flowing over aerated sharp-crested weirs The general form of the sheet is indicated in Fig 16 of Chapter 8 If the area below the lower nappe is filled with masonry, the shape of the sheet and the discharge will not be changed appreciably.

This shape of crest has become standard in modern designs except under special circumstances. For reference, therefore, it will be designated the "standard dam crest," and the head on the crest, used in establishing it, will be termed the "design head"

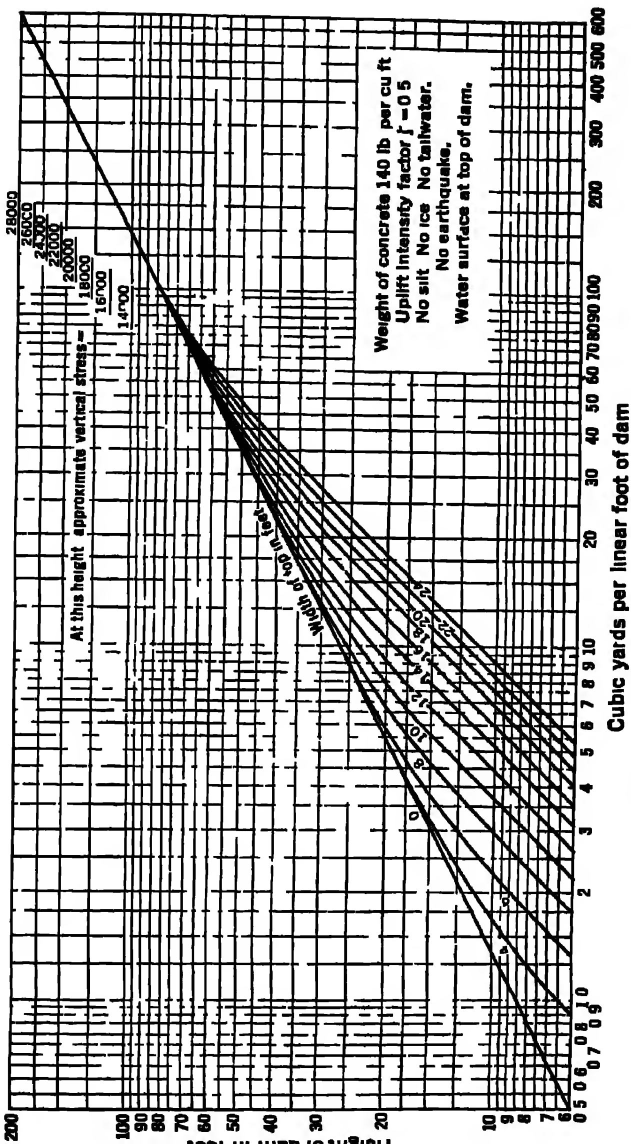
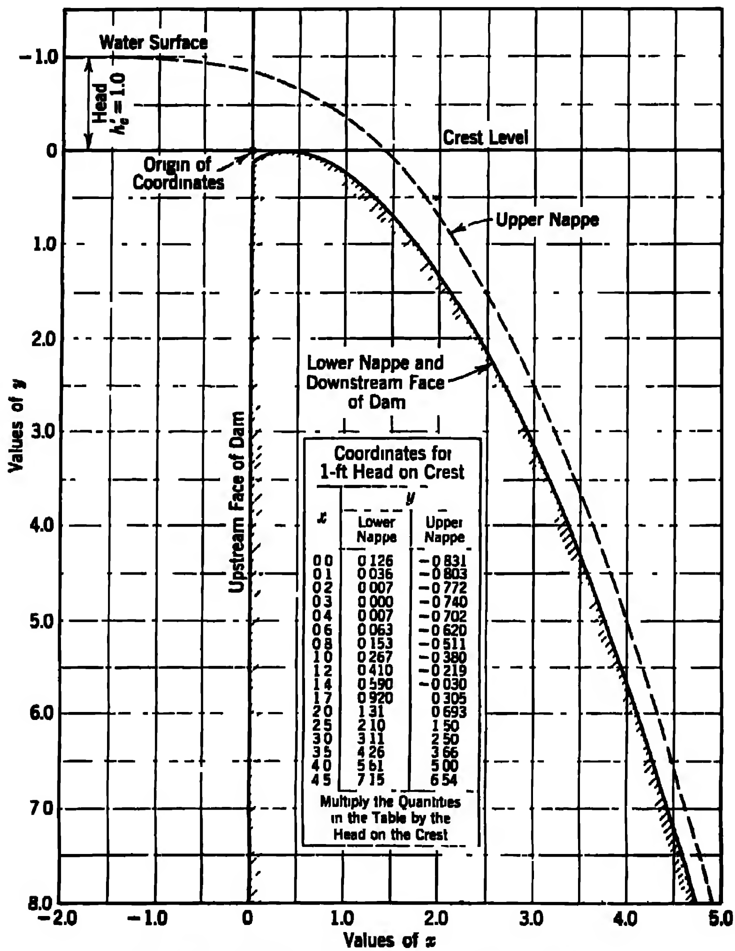


Fig 19 Contents of solid non-overflow dams

Figures 20 and 21 give recommended coordinates* of the nappes and shape of crest for dams with vertical face and with face inclined on a 45-degree



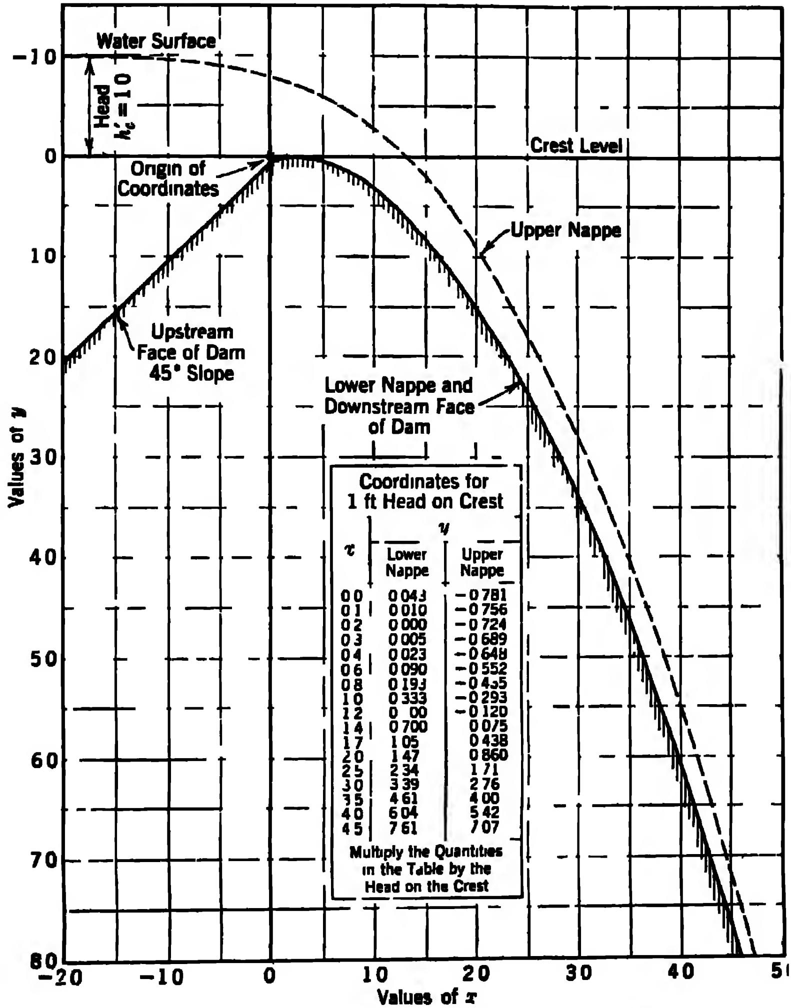
x	y: Lower Nappe	y: Upper Nappe
0.0	0.126	−0.831
0.1	0.036	−0.803
0.2	0.007	−0.772
0.3	0.000	−0.740
0.4	0.007	−0.702
0.6	0.063	−0.620
0.8	0.153	−0.511
1.0	0.267	−0.380
1.2	0.410	−0.219
1.4	0.590	−0.030
1.7	0.920	0.305
2.0	1.31	0.693
2.5	2.10	1.50
3.0	3.11	2.50
3.5	4.26	3.66
4.0	5.61	5.00
4.5	7.15	6.54

FIG. 20. Standard crest, upstream face vertical.

slope, respectively, and with no velocity of approach. The coordinates in the tables are to be multiplied by h'_c to get the coordinates of the concrete crest. For appreciable velocity of approach, the shape of the crest can be approximated safely, for preliminary designs, by multiplying the coordinates in the

* William P. Creager, *Masonry Dams*, John Wiley & Sons, 1929, p. 106.

table by h_c' plus the velocity of approach For the exact shape of the crest with a velocity of approach see p 362 of Ref 1



Coordinates for 1 ft Head on Crest

x	y Lower Nappe	y Upper Nappe
0 0	0 043	−0 781
0 1	0 010	−0 756
0 2	0 000	−0 724
0 3	0 005	−0 689
0 4	0 023	−0 648
0 6	0 090	−0 552
0 8	0 193	−0 435
1 0	0 333	−0 293
1 2	0 [illegible]00	−0 120
1 4	0 700	0 075
1 7	1 05	0 438
2 0	1 47	0 860
2 5	2 34	1 71
3 0	3 39	2 76
3 5	4 61	4 00
4 0	6 04	5 42
4 5	7 61	7 07

Multiply the Quantities in the Table by the Head on the Crest

FIG 21 Standard crest upstream face inclined 45°

Experiments by Rouse and Reid [10] indicate that the shape of the crest will not be affected materially by details shown in Fig 22a and 22b, provided the distance, d is equal to at least one half the sum of the head on the crest and the head corresponding to the velocity of approach This is because vertical velocities are small below such depth

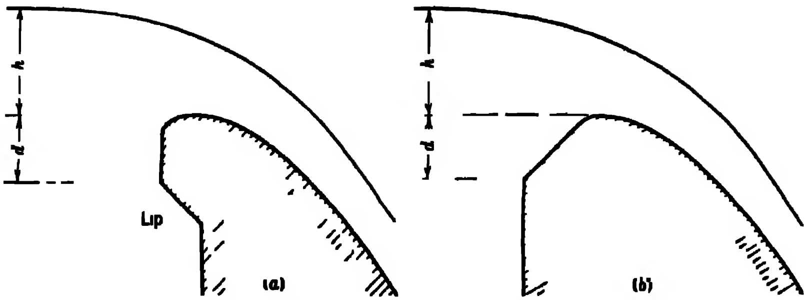


Fig 22 Special crest details

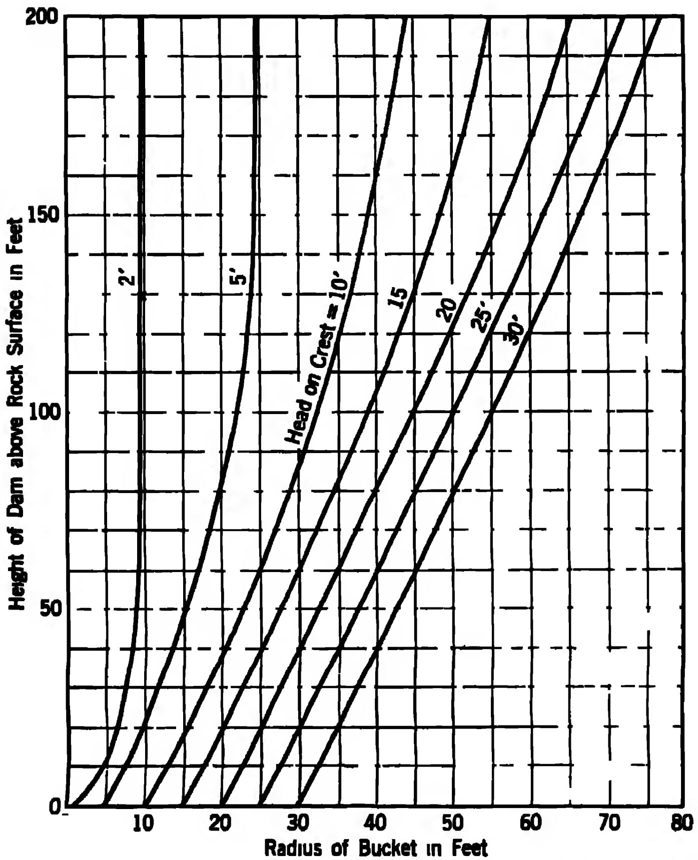


Fig 23 Recommended radius of bucket for spillway dams with no tailwater

Experiments [10] on models of standard dam crests have shown that the jet adheres to the concrete for heads on the crest up to 2 or 3 times the design head, even though the most effective practical means for admitting air is provided. Although it is not certain that the relationship will hold directly between model and prototype, adherence of the jet to accurately designed crests can be expected for heads considerably higher than the design head.

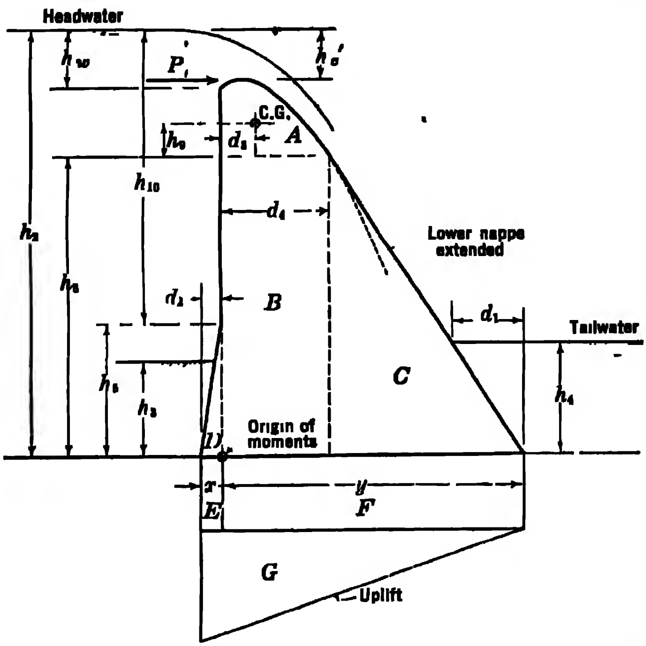


FIG. 24 Diagrammatic sketch explaining single-step design for spillway dams

Should the crest contain any irregularities that would cause eddies, or should it not correspond to the shape of the jet for appreciable velocities of approach, as previously explained, negative pressures may be set up on the face of the dam.

Since the adhering nappe with the resulting partial vacuum under it increases the coefficient of discharge, the crest, in special cases of spillway dams without gates on the crest, has been shaped for a design head somewhat less than that for the expected flood in order to increase the spillway capacity. At the same time provisions are made for resisting the vacuum load on the dam. The difference in the discharge coefficient with and without adhering jets is shown in Fig. 15 of Chapter 8

34. The Bucket. Except for low dams with small maximum discharges and the best rock foundations, spillway dams should have a fillet or "bucket" at the toe to deflect the sheet of water in a horizontal direction. A usual

TABLE 3

BASIC EQUATIONS FOR SOLID SPILLWAY DAMS

(SINGLE-STEP DESIGN)

Line	*Item*		*Force*	*Lever*
	VERTICAL FORCES			
1	Concrete	Portion *A*	$+w_1$ (Area *A*)	$+d_3$
2	"	− Rectangle *B*	$+w_1h_8d_4$	$+\frac{d_4}{2}$
3	"	Triangle *C*	$+\frac{w_1h_8y}{2}-\frac{w_1h_8d_4}{2}$	$+\frac{y}{3}+\frac{2d_4}{3}$
4	"	− Triangle *D*	$+\frac{w_1h_5x}{2}$	$-\frac{x}{3}$
5	Headwater		$+w_2h_{10}x$	$-\frac{x}{2}$
6			$+\frac{w_2h_5x}{2}$	$-\frac{2x}{3}$
7	Tailwater		$+\frac{w_2h_4d_1}{2}$	$+y-\frac{d_1}{3}$
8	Silt *		$+\frac{w_1h_3^2x}{2h_5}$	$-x\left(1-\frac{h}{3l}\right)$
9	Uplift	−Rectangle *E*	$-w_2h_4x$	$-\frac{x}{2}$
10	"	Rectangle *F*	$-w_2h_4y$	$+\frac{y}{2}$
11	"	Triangle *G*	$-w_2c(h_2-h_4)\left(\frac{x}{2}+\frac{y}{2}\right)$	$+\frac{y}{3}-\frac{2x}{3}$
12			ΣW	
	HORIZONTAL FORCES			
13	Headwater		$+\frac{w_2h_2^2}{2}$	$+\frac{h_2}{3}$
13*A*			$-\frac{w_2h_{10}^2}{2}$	$+h_2-\frac{2h_{10}}{3}$
14	Ice †			
15	Tailwater		$-\frac{w_2h_4^2}{2}$	$+\frac{h_4}{3}$
16	Silt *		$+\frac{w_3h_3^2\sigma}{2}$	$+\frac{h_3}{3}$
	EARTHQUAKE			
17	Concrete	Portion *A* ‡	$+\alpha$(Line 1)	$+h_8+h_9$
18	"	Rectangle *B* ‡	$+\alpha$(Line 2)	$+\frac{h_8}{2}$
19	"	Triangle *C* ‡	$+\alpha$(Line 3)	$+\frac{h_8}{3}$

TABLE 3—*Continued*

BASIC EQUATIONS FOR SOLID SPILLWAY DAMS

(SINGLE-STEP DESIGN)

Line	*Item*	*Force*	*Lever*
20	Concrete—Triangle *D* ‡	$+\alpha$(Line 4)	$+\frac{h_5}{3}$
21	Water §	$+\frac{2C_e a h_2^2 p_f}{3}$	$+\frac{0.4 h_2 p_m}{p_f}$
22	Ice	Ignored	
23	Silt	Ignored	
24		ΣP	ΣM

* $\sigma = \frac{1 - \sin\phi}{1 + \sin\phi}$ from Eq. 8. Lines 8 and 16 are for full reservoir. Use saturated silt w_5 for empty reservoir.

† Not used for high water at spillway dams. See Section 8.

‡ Use negative force for reservoir empty.

§ Where p_f and p_m are as indicated in Part *d* of Section 14.

type of bucket is indicated in Fig. 30 of Chapter 8. Its purpose is obviously to prevent the impact of the falling water from scouring the foundation at the toe of the dam. Suggested dimensions are shown in Fig. 23 of this chapter.

Should the foundation be of such a character that, even with a bucket, some scour from the spilling water can be expected, more extensive provisions to prevent scour must be made. Section 5 of Chapter 25 describes these provisions.

To be thoroughly effective the bucket should be tangent to the foundation, or nearly so. A sudden enlargement of the jet, due to a vertical step, where it joins the foundation will cause an eddy that, under high velocity, will erode a stratified or soft foundation. The bucket is not considered in computing stability of the dam.

35. Application of the Single-step Design to Spillway Dams. The first step in the design of spillway dams is to shape the top of the dam to fit the lower nappe of the jet corresponding to the design flood head on the crest, as indicated in Fig. 24 and explained in Section 33. Select a point on the lower nappe where it has an inclination of 30 degrees with the vertical, and draw a line from that point to the base, intersecting the base at the downstream extremity of the assumed dimension y. This line will not be tangent to the lower nappe but will be near enough for practical purposes. Other dimensions and methods of procedure follow the explanation in Part E.

36. Basic Equations for Single-step Design. The basic equations given in Table 3 will be found useful in the design of solid spillway dams by the single-step method. Nomenclature is given in Section 3 with additions in Fig. 24.

Moments are taken about the base of the section, on a vertical extension of the vertical top part of the upstream face. Clockwise moments are positive. Downward vertical forces and horizontal forces directed to the right are also positive.

It may be noted that, unless the base pressures are excessive so that the base of the dam has to be widened to reduce them, the upstream face of spillway dams need not be battered in order to keep the resultant, reservoir empty, within the middle third, in conformity with Rule 1 of Section 18.

37. Example 5. 100-ft Spillway Gravity Dam. Let it be required to estimate the approximate cross-section of a spillway gravity dam 100 ft in

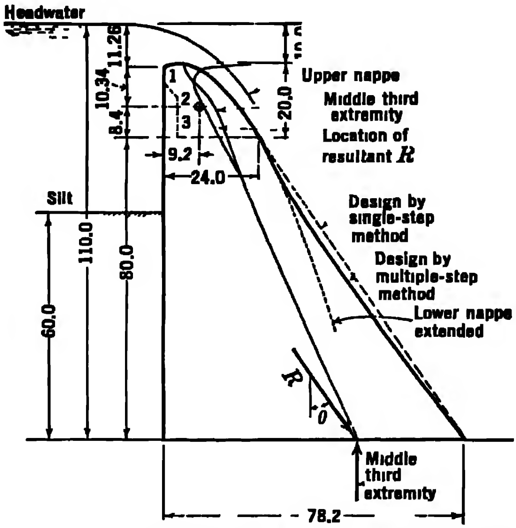


FIG. 25. Elements of the section for Example 5.

height on a good foundation. It is assumed, for this case, that the design is governed by high water and no ice, instead of low water and ice pressure.

Since the base pressures do not govern, the upstream face, as mentioned in Section 36, will not have to be battered and the value of x is zero. These circumstances simplify the solution because, with only one variable, y, its derivation is direct. Referring to Fig. 25, let

$h_2 = 110$ ft;	$d_3 = 9.2$ ft;
$h_c' = 10$ ft;	$d_4 = 24$ ft;
$h_u = 10 \times (1 + 0.126)$	$w_1 = 150$ lb per cu ft;
$= 11.26$ ft (Fig. 20);	$w_2 = 62.5$ lb per cu ft;
$h_8 = 80$ ft;	$\zeta = 0.5$;
$h_9 = 8.4$ ft;	Area $A = 330$ sq ft.

Also, let there be no tailwater and no silt.

TABLE 4

Computations for Example 5

(100-ft Spillway Dam—Reservoir Full)

Line	Item	Force	Lever	Moment
VERTICAL FORCES				
1	Concrete Portion A	$+150 \times 330 = +49{,}500$	$+9.2$	$+455{,}000$
2	" Rectangle B	$+150 \times 80 \times 24 = +288{,}000$	$+\frac{24}{2}$	$+3{,}456{,}000$
3	" Triangle C	$\left.\begin{array}{l} +150 \times 80y/2 = +6000y \\ -150 \times 80 \times \frac{24}{2} = -144{,}000 \end{array}\right\}$	$+y/3 + \frac{2 \times 24}{3}$	$+2000y^2$ $-48{,}000y$ $+96{,}000y$ $-2{,}304{,}000$
4	" Triangle D	None as $x = 0$		
5	Headwater	None as $x = 0$		
6	Headwater	None as $x = 0$		
7	Tailwater	None		
8	Silt	None		
9	Uplift Rectangle E	None as no tailwater		
10	Rectangle F	None as no tailwater		
11	" Triangle G	$-62.5 \times 0.5 \times 110 \times y/2 = -1720y$	$+y/3$	$-573y^2$
12		ΣW_F		
HORIZONTAL FORCES				
13	Headwater	$+62.5 \times 110^2/2 = +378{,}000$	$\frac{110}{3}$	$+13{,}860{,}000$
13.4		$-62.5 \times 11.26^2/2 = -3960$	$+110 - 2 \times \frac{11.26}{3}$	$-406{,}000$
14	Ice	None		
15	Tailwater	None		
16	Silt	None		
17–23	Earthquake	None		
24		ΣP_F		ΣM_F

For Reservoir Full

$$\Sigma W_F = 193{,}500 + 4280y \qquad [39]$$

$$\Sigma M_F = 15{,}061{,}000 + 48{,}000y + 1427y^2 \qquad [40]$$

To find the value of y such that the resultant will intersect the base at the extremity of the middle third, we have, from Eq. 38,

$$\Sigma M_F = \Sigma W_F \tfrac{2}{3} y \qquad [41]$$

or

$$15{,}061{,}000 + 48{,}000y + 1427y^2 = \tfrac{2}{3}y(193{,}500 + 4280y)$$
$$= 129{,}000y + 2850y^2$$

Substituting the values of ΣW_F and ΣM_F, from Eqs. 39 and 40, in Eq. 41, we find that $y = 78.2$.

Figure 25 shows the resulting section and, for comparison, a section derived by the multiple-step method. It will be noticed that, for this example, the width of the base of the dam is practically the same with either method.

This is because the center of gravity of the area added by the single-step method is almost directly over the point of intersection of the resultant R with the base and therefore has no effect on its location.

Tests for conformity with the other rules of design are the same as those described for Example 3, Section 28.

Near the top of spillway dams the resultant R will fall well outside the extremity of the middle third, as shown in Fig. 12*b*, and tension will exist in the concrete in violation of Rule 1 of design indicated in Section 18. However, such tension is almost universally ignored even when ice pressure is involved. For this example without ice, the tension exists for a distance of about 4 ft below the crest.

Also, near the top of spillway dams, the angle θ of inclination of the resultant becomes greater than the allowable value without consideration of shearing strength in the concrete, and Eq. 25 applies. However, it will be found that the shearing strength of concrete will be ample, even with considerable ice pressure.

Therefore, the effect of ice pressure need be considered only in respect to the base of the dam where tension cannot be counted on.

Assuming water surface at the crest of the dam and 10,000 lb per lin ft of ice at crest level, it will be found that low water and ice does not govern the design for the section of the dam that is 100 ft high; in fact, it would not govern, in this example, for any section higher than about 30 ft For sections less than 30 ft high, where low water and ice does govern, a heavier section can be adopted by one of two methods.

The first method is to adopt an arbitrary head, h_c', on the dam, greater than the actual value of the design head and shape the crest accordingly This method has the disadvantage of decreasing the coefficient of discharge, since the design head would be less than the head for which the crest is shaped.

The second method is to add additional concrete to the upstream face and shape the crest for the design head according to Fig. 21, as indicated in Fig. 22*b*. In many cases, this will require the lesser quantity of concrete.

If it is desired to have the entire length of the dam of the same shape, the section thus required for heights less than the 30 ft can be used as the start of the design for the highest or 100-ft height, as previously explained.

38. Example 6. 20-ft Spillway Dam. Let it be required to design a dam under the same conditions as for Example 5 but only 20 ft high. No ice thrust is assumed.

The top 20 ft of Fig. 25 can be used for this example. Proceeding as indicated for Example 5, we will find that the resultant for both full and empty reservoir intersects the base well within the middle third. If it should be found that the sliding factor on the foundation is also safe, it is evident that, except for the requirement regarding the shape of the downstream face, the section can be reduced. This condition can be met by undercutting the up-

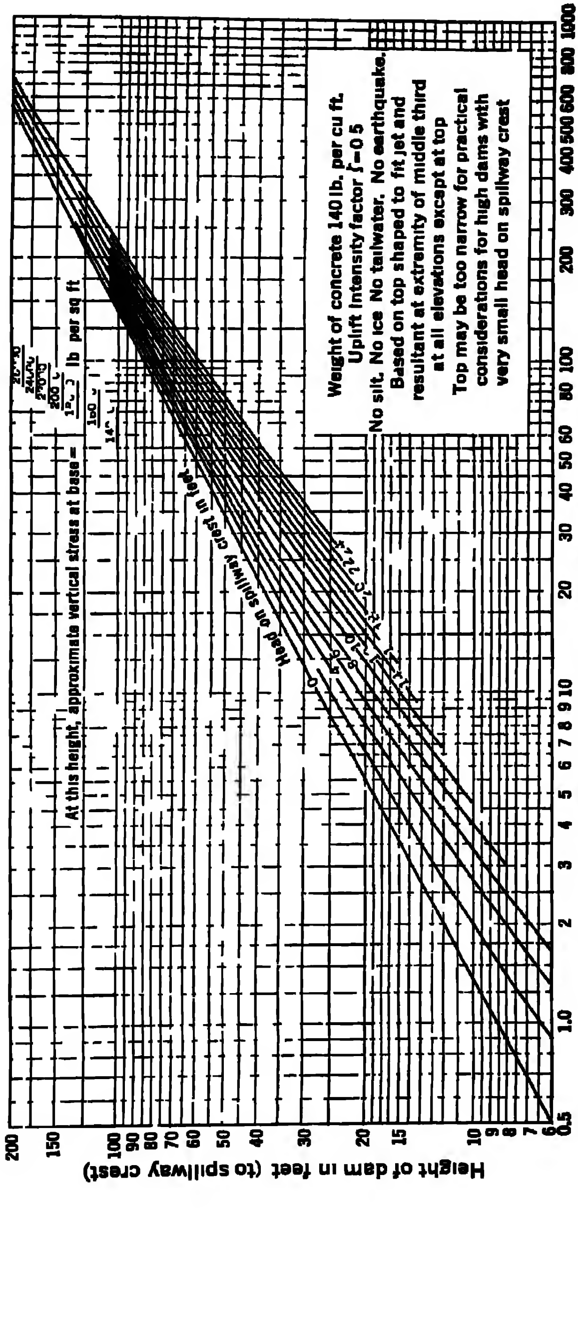


FIG 26 Contents of solid gravity spillway dams

stream face, as illustrated in Fig. 22a and as indicated by dotted lines 1-2-3 of Fig. 25.

39. Ice on Controlled Crests. If crest gates are likely to be closed and the reservoir full to flood level during very cold weather, the importance of ice pressure may be greatly increased. The gates themselves are subject to damage and the ice thrust is raised to a higher level, increasing the overturning effect on the dam if the gates are sufficiently strong to transmit the thrust.

40. Approximate Quantities for Spillway Gravity Dams. Figure 26 shows approximate quantities of concrete in spillway dams at which the foundations are such that the allowed tan θ (Fig. 9) is at least 0.76 and the strength of the foundations is sufficient to withstand the stresses. An uplift intensity of 0.5 is assumed. The weight of concrete is 140 lb per cu ft.

The use of this diagram where concrete actually weighs 150 lb per cu ft will give a degree of conservatism of about 4% for high spillway dams, gradually diminishing to zero for low spillway dams in which the shape of the section is governed by the shape of the jet of spilling water, with no undercutting of the upstream face as indicated in Section 38.

G. CONCRETE FOR DAMS *

41. General. The structural quality of concrete is so closely related to the structural quality of the aggregate of which it is composed that it is hazardous to conclude that, if the cement and aggregate tested alone prove durable, the concrete will be equally durable. There are too many examples to the contrary, and hence a thorough and comprehensive series of tests of all ingredients alone, and also in the proposed mixes, is mandatory for important structures if later failures are to be avoided. The testing of concrete ingredients is a good deal like the examination of a dam site—rarely is it sufficiently exhaustive to leave no regrets.

The specifications for the concrete in a structure should be comprehensive but at the same time should be couched in generalizations rather than in too much detail. Results rather than methods should be specified, if practicable. And then, an experienced and well-qualified inspection force should be provided and clothed with the authority necessary to secure the desired results.

42. Cement. Since cement is the least stable ingredient in concrete, it has an important influence on the life of a concrete structure, especially a hydraulic structure in a climate where a large number of alternate freezings and thawings annually result in progressive surface disintegration.

Although natural and Portland-pozzolana cement are sometimes used in the construction of dams, the vast majority of dams are built with Portland cement. American Society for Testing Materials Specifications are available

* Extracted from Chapter 15, *Engineering for Dams*, John Wiley and Sons, New York, 1945, by Byram W. Steele, Head Engineer, Office of Chief of Engineers, War Department, Washington, D. C.

for five types of Portland cement—I, general purpose; II, moderate heat; III, high early strength; IV, low heat; and V, sulphate resistant. Federal specifications also are available on these cements.

The tricalcium aluminate content of all five types is limited as follows: I, 15%; II, 8%; III, 15%; IV, 7%; and V, 5%.

Type II, commonly known as modified cement, is becoming quite popular for hydraulic structures exposed to moderate sulphate action, or where moderate heat of hydration is an essential feature, and where it is desired to specify certain chemical and physical test requirements.

Type IV, low-heat cement, is used for massive concrete work in which it is desired to limit the maximum temperature in the concrete to the smallest value practicable and to produce as crack-resistant a concrete as possible.

Type V cement is used where specially resistant qualities are necessary on account of severe alkali conditions.

43. Fine Aggregate. The gradation, or particle-size distribution, of the sand has an important bearing on the workability, durability, and cement content of concrete. The allowable grading limits depend to some extent on the shape and surface characteristics of the particles. For example, a sand having smooth, well-rounded particles will give satisfactory results with a much coarser grading than a manufactured sand with angular particles. The fineness modulus of sand, which is the sum of the percentages retained on the number 4, 8, 16, 30, 50, and 100 Standard sieves divided by 100, should, for general concrete work, be maintained between 2.50 and 3.00, and the variation from the average on any job should be held to plus or minus 0.1 if uniformity and close control of placement are desirable.

44. Coarse Aggregate. The number 4 screen, which has a square opening of 3⁄16 in., is generally used as the dividing point between fine and coarse aggregates. For mass concrete in dams the maximum size of stones incorporated in the concrete has gradually decreased, until now 6 in. has been generally adopted as the largest cobble practicable to put through the mixer and hence the largest particle of rock that can be economically incorporated in the concrete. To avoid undesirable segregation, coarse aggregate should be divided into 2, 3, or 4 sizes, depending on the maximum size of the cobble permitted. The selection of screens to produce these sizes depends on the natural gradation of the coarse aggregate. A common size division is 3⁄16 to 3⁄4 in., 3⁄4 to 1½ in., 1½ to 3 in., and 3 to 6 in. The dividing point between sizes is more or less arbitrary, but in some cases it is necessary to modify the usual size limitations to prevent too much difference in size of batching units.

45. Water. The water used in concrete, mortar, and grout must be reasonably clean and free from objectionable quantities of silt, organic matter, alkali, salts, and other impurities.

46. Admixtures. If concrete to withstand severe weathering conditions is desired, an air-entraining admixture will improve the durability of the concrete and also its placeability. To obtain the most benefit from an air-en-

training agent, the gradation of the sand and the design of the mix should be given special study by a competent concrete technician.

47. Concrete Mixes. The designing of the concrete mixes for a dam involves the combination of available materials so as to produce concrete of the desired durability, impermeability, and strength at minimum costs. Theoretical mix-design methods found in the literature may well be followed in designing concrete mixes, but these methods are often too complicated for general use; so it is advisable to select trial mixes which will approximate the desired characteristics and then adjust these trial mixes to suit local aggregates and conditions. For details of this procedure reference is made to the report of Committee 613 in the November 1943 *Journal of the American Concrete Institute* and to *Recommended Practice for the Design of Concrete Mixes* (ACI 613–44). The Trial Mix design procedure given in the committee report is an excellent approach to this subject and one of the most direct and simple yet devised.

In a massive structure, economy dictates the use of the maximum size of aggregate that is available and can be handled advantageously through the mixers. If the structure, however, is of the multiple-arch or slab-and-buttress type, the thickness of the members and the reinforcement details generally dictate the maximum size of aggregate that can be used to advantage, and durability and impermeability rather than strength will be the factors that will influence the selection of the cement content.

48. Batching and Mixing. In the batching and mixing plants now in general use for dam construction, the aggregate is delivered to the top of the storage bins by belt conveyor and the cement is pumped through pipes. The storage bins are above the batchers and the mixers below them. All materials descend through the plant and into the buckets by gravity. Batching equipment is usually air-operated and electrically controlled from a single board. In the latest plants the tilting mixers are charged from a common central collecting cone and discharged through a central common hopper into the bottom-dump buckets.

The tilting mixer is the only type that will satisfactorily handle mixes containing cobbles. The shape of the mixer drum and the shape and location of the blades within the drum, as well as the method and sequence of charging, have a marked effect upon the uniformity of the batch. All mixers should be so located and arranged as to permit the operater to view the mixing operation during its progress rather than to judge the qualities of the mix after it is dumped. The proper mixing time for any batch depends on the speed of rotation of the mixer and the other factors mentioned above; it varies from 1 min for small mixers to 2½ to 3 min for 4-yd mixers.

49. Transportation and Placing. The trestle and the cableway are the most common systems of distributing concrete over the structure. If a cableway is used, bottom-dump buckets are transported from the mixer to the cableway on cars; and the cableway, in order to serve any point on the dam, is equipped with a movable tower at one or both ends, depending on the

plan layout of the dam. If a trestle is used, the cars transport the buckets from the mixer along the trestle to the point of deposit, where a crane picks the bucket off the car and spots it in the forms.

Concrete should be transported from the mixer as a unit mixture, deposited as near as practicable in its final position, and consolidated by vibration with as little segregation as possible. The vibrator by permitting the satisfactory placement of concrete too dry to place by hand has done more than any other agency to promote the benefits of the water-cement ratio law.

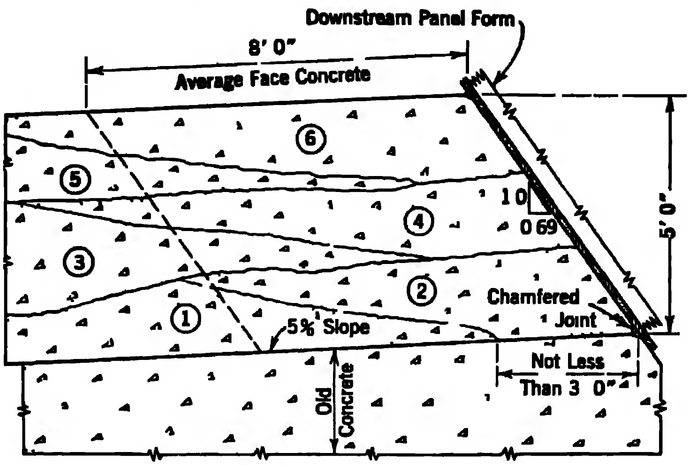


FIG. 27. Sequence of pours (circled numerals) for concrete on downstream face of dam. (O. J. [illegible], in *Proc.* A.S.C.E., March 1941.)

In massive concrete dams the exterior shell generally contains more cement than the interior, and a common stipulation relative to the control and placing of this face mix is as follows: The concrete to a depth of 5 ft normal to the face shall contain more cement per cubic yard than the concrete in the interior of the dam. It shall be placed as nearly simultaneously with the adjacent interior mix as plant operations will permit so that the two mixtures will unite in their plastic state to form an integral mass. The last clause contemplates an overlapping and dovetailing of the batches as they are dumped and consolidated by vibration. For a schematic picture of this concrete face-mix placing operation see Fig. 27.

50. Forms and Formed Surfaces. Tongue and grooved narrow timber lagging for ordinary surfaces outranks any other form material since it permits the escape of more air and water bubbles than sheet metal and such materials as masonite, thus eliminating much objectionable sand streaking and pitting. Whenever a dense durable pock-free matt surface is desired some form of absorptive form lining such as Celotex, Firtex, or U. S. Rubber Company lining should be used. For parapets, penthouses, retaining walls and the exterior and interior of structures to house the control works an

absorptive form lining produces an ideal surface, if due care is taken in the building of the forms and the placing and consolidation of the concrete. In effect, it case-hardens the concrete at the surface by straining out the air and water, holding back the cement, and thus producing more intimate particle contact and increased impermeability.

In the construction of spillway buckets, ogee crests, and other structures, slopes steeper than 1 on 1¾ should be formed. Concrete properly placed against a formed surface is more resistant to the elements and to erosion than a surface that has been disturbed by screeding, floating, and troweling.

51. Height of Lifts. In order to control the maximum temperature in mass concrete, it is generally specified that it shall be poured in 5-ft lifts, but multiple-arch dams, slab-and-buttress dams, and retaining walls are ordinarily carried up in 10-ft, 12-ft, or even higher lifts, since the dissipation of excess heat from such structures and the prevention of cracks are more readily accomplished. Lifts higher than 5 ft have been used in mass concrete, but the formwork difficulties increase rapidly with height for mass-concrete types of forms. From a purely construction standpoint, lifts higher than 5 ft may be desirable for mass-concrete work but, after all factors entering into this question have been given due consideration, the 5-ft lift seems preferable on most mass-concrete jobs. One of the objections to high lifts, especially in the arches and face slabs of dams, is the tendency toward the accumulation of water and cement at the top of the lift, thus producing a thin band of high water-cement-ratio, porous, nondurable concrete. This condition in high lifts can be avoided and satisfactory concrete secured, but to do so requires eternal vigilance on the part of the inspection force.

52. Curing and Protection. Vertical surfaces can be cured very satisfactorily by arranging a system of pipes with spray nozzles at such intervals that the entire surface is covered with a fine spray. For horizontal surfaces the most positive means of securing continuous moist-curing is a blanket of saturated sand applied immediately after placement is completed. This method of curing results in a surface that is as near ideal for starting the next lift as it is possible to produce and is equally good for finished surfaces.

Curing compounds, both colored and colorless, are permitted on many jobs, and when they are properly applied the degree of perfection of the curing is probably as good as the average water-cured job, but it is universally recognized that no curing compound will give the equivalent of *continuous* moist-cure. Colored compounds, to be most effective in reducing surface cracking, should be covered with a coat of whitewash so that the heat of the sun will be reflected instead of absorbed. Moisture, in addition to that present as mixing water, will be taken up by the cement if this excess moisture is readily available at the right time.

Winter concreting involves protection against freezing until the concrete has attained sufficient strength to permit normal construction operations without damaging the concrete. This length of time varies according to the temperature of the mix when placed and the kind of cement used. One of

the chief objections to low-heat cement for dams in northern climates is the increase in time between lifts necessary to accommodate its slower-setting characteristics.

53. Joints—Horizontal and Vertical. Surface clean-up is best accomplished by the use of a high-velocity jet of water and air applied at the proper time during the setting period. Once the concrete is hardened, clean-up is best accomplished by the use of the wet-sand blasting process, which will remove any undesirable material rather effectively and economically. Regardless of what kind of treatment the joint receives before another lift is started, ½ in of mortar should be applied immediately before concrete placing begins, to permit the proper bedding of the aggregate in the fresh concrete and the proper bond of old and new concrete.

54. Temperature Control, Cracking. The following are some of the various operations that may be incorporated into the construction program for a dam as a practical solution of the elimination of cracks: (1) starting off all rock foundations or concrete surfaces that have set for several weeks with two or three 2½-ft lifts and 5 days between lifts; (2) limiting the height of all other lifts to 5 ft and 5 days between lifts; (3) sprinkling the coarse aggregate and blowing compressed air through it in summer to standardize the moisture content and reduce the temperature (this operation at Hiwassee Dam reduced the temperature about 3 degrees); (4) refrigeration of the mixing water, including use of ice if it can be properly batched and discharged into the mixer; (5) use of low-heat cement; (6) use of low cement content—0.8 bbl per cu yd for interior and 1.0 bbl for exterior or surface shell. If the gradation and particle shape of the aggregate permit, 3 sacks of cement per cu yd can be successfully used for interior concrete in a massive dam; (7) circulating cold water or ice water through pipes imbedded in the concrete of each lift as soon as the pipes are covered and until the temperature of the mass has been reduced to mean annual temperature for that locality; (8) control of form removal so that high differentials in temperature between the surface and near-surface areas do not take place. The size of the dam, the amount of concrete, and the form in which the concrete is placed will be important factors in determining how many of these operations are applicable.

55. Waterproofing. If well-graded aggregates are available and the mixture is properly designed, mixed and placed, and then adequately cured, concrete can be made tight for all practical purposes even under high pressures. However, cold joints between pours, settlement or shrinkage cracks, careless placement, segregation, laitance bands at the tops of lifts, and other defects are the common sources of leakage which make waterproofing necessary in relatively thin concrete structures having one side exposed to direct water pressure when the other side must be absolutely dry.

Bituminous coatings, oil paints, oil resin combinations, Portland cement paint, powdered iron preparations, and other proprietary surface treatments have been used with varying degrees of success. Bituminous membrane

waterproofing has been used extensively in Europe in connection with hydroelectric projects.

Integral waterproofing should also be mentioned; but the use of powdered admixtures to improve watertightness is limited to the leaner mixes, in which the additional fines increase the workability and thus make possible a more dense and impervious mass.

56. Tests of Concrete and Concrete Materials. Owing to the unsatisfactory service record of many structures, the tendency today is toward closer inspection and an increase in the types of tests conducted on concrete and the ingredients of concrete. The American Society for Testing Materials has available standard methods for all such tests, and the 1940 report of the Joint Committee on Standard Specifications covers in detail Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete.

57. Bibliography. Some references are mentioned in the text of this chapter; others serve to give additional data on the subject.

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CHAPTER 18

ARCH DAMS

1. Types of Arch Dams. Dams having all or a part of the structure in the form of an arch may be divided into three general types, viz., single-arch, multiple-arch, and arched-gravity dams. Single-arch dams are treated in this chapter. Multiple-arch dams, consisting of a series of arches supported by buttresses, are described in Chapter 19. Arched-gravity dams are essentially

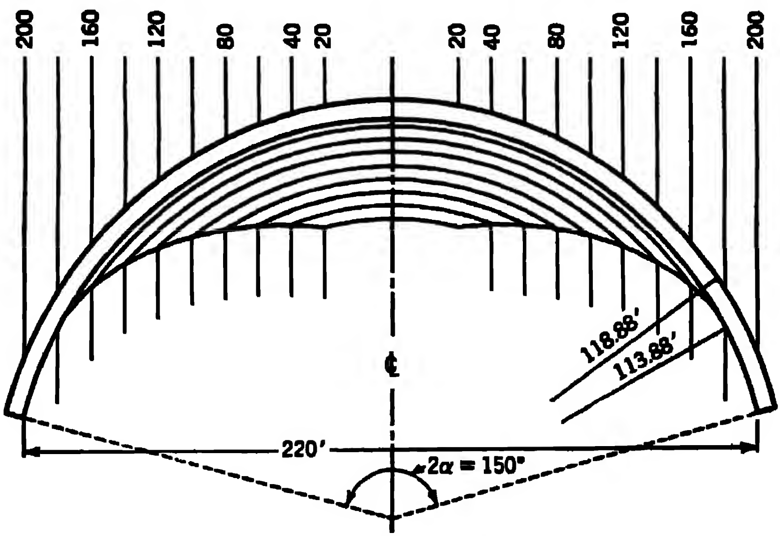


FIG. 1 Constant-radius arch dam.

gravity dams curved in plan Such dams are beyond the scope of this book and, for preliminary investigations, may be treated as straight gravity dams.

The single-arch dam is adaptable when the length is small in proportion to the height, and when the sides of the valley are composed of good rock which can resist the end thrust. Under favorable conditions, it contains less material than other concrete types and, being equally permanent, is usually adopted where conditions permit Unfortunately, few sites are suitable for this type of dam.

The single-arch dam may be designed as a constant-radius or a constant-angle arch dam. These types are shown in Figs. 1 and 2 respectively. For the constant-radius dam, the central angle 2α is necessarily the greatest at the top of the dam and reduces greatly towards the bottom. For the constant-angle dam, the central angle 2α is constant and the radius is greatest

at the top of the dam, reducing towards the bottom. The constant-angle arch dam has been shown to be by far the most economical and has been adopted for modern dams. However, where the site is not as regular as it is in Fig. 5, the central angle may have to vary somewhat to suit local conditions.

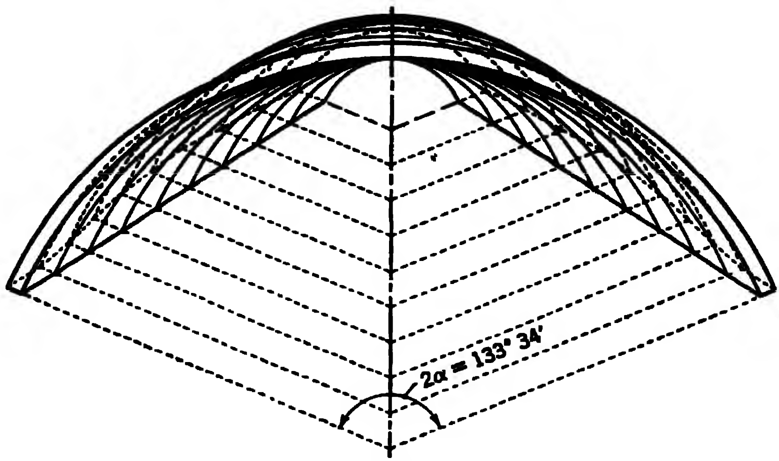


FIG 2. Constant-angle arch dam.

2. Nomenclature. In the following, feet and pound units are intended.

C_F = coefficient of thermal expansion and contraction of concrete.
d = location of resultant at abutments.
e = eccentricity of resultant at abutments.
E = modulus of elasticity of concrete in pounds per square inch.
F = temperature drop in concrete in degrees Fahrenheit.
h = depth of water at any elevation.
l_i = chord length of arch at the intrados.
p = unit compressive stress by cylinder theory.
p_1 = arch load compressive stress at abutment by elastic theory.
p' = 28-day compressive strength of concrete.
p_t = temperature compressive stress at abutment.
P = allowed maximum compressive stress.
r_e = radius of extrados of arch.
r_i = radius of intrados of arch.
r = mean radius of arch.
R = a constant representing the ratio p_1/p.
R' and R'' = horizontal and inclined total abutment thrust, respectively.
t = thickness of arch at any elevation.
w_2 = weight of water = 62.5.
α = one half the central angle of the arch in degrees.
β = angle between abutment thrust and a normal to the radius.
ϕ = angle of friction.

3. Cylinder Method. In the cylinder method of design, stresses are computed on the basis of an unrestrained, thin cylinder of the same radius as the arch. Under this method, the unit stress in the arch may be obtained from:

$$p = \frac{w_2 h r_e}{t} \qquad [1]$$

However, the cylinder method does not apply directly to arch dams for the following reasons. All dimensions of an elastic unrestrained cylinder reduce under load. But the arch dam is not a complete cylinder. The length of the arch reduces under load, but the abutments are fixed and the span l_t remains constant. This causes bending moments in the arch with increased unit compressive stresses. For arch dams, particularly where the ratio t/r is large or when the central angle 2α is small, the resulting increase in stresses is considerable. For this reason either the elastic arch method or the trial-load method of design should be used for final design purposes.

4. Elastic Arch Method. The water load on an arch dam is supported not only by arch action, which transfers some of the load horizontally to the abutments, but also by cantilever action which transfers some of it vertically to the foundation. The division of the load between the arches and the cantilevers cannot be determined readily and, if considered acting simultaneously, must be analyzed by the laborious trial-load method explained later.

The following adaption of the elastic arch theory, for preliminary designs, ignores the assistance given by the vertical cantilevers. It also makes no allowance for the factors of shrinkage of concrete, plastic flow, moisture-volume change, Poisson's ratio, and yield of abutments, but considers stresses due to direct load and temperature changes only.

Ignoring the assistance given by the vertical cantilevers is on the conservative side for the lower part of the dam but may actually increase the arch stresses near the top of the dam, which is subject to stress transfer by the action of the cantilevers. However, this feature is compensated for by extra thickness given to the upper arches, as explained in Section 9. Lubricated horizontal joints [9], lately placed in arch dams close to the foundations to eliminate horizontal shear due to cantilever action, have proven successful in at least partially preventing the effect of cantilever action.

Stresses up to 25% of the 28-day strength of the concrete, but not exceeding 800 lb per sq in., have been used in modern arch dams designed by more exact analysis, as explained in Chapter 13 of *Engineering for Dams* [1]; but, for preliminary designs, made in accordance with the methods outlined herein, 22% of the 28-day strength, but not exceeding 700 lb per sq in., is recommended for symmetrical sites, with somewhat lower stresses for unsymmetrical sites.

The following adaption of the elastic arch theory is based on equations developed by Professor William Cain [6, p. 522] for fixed-end arches and including the influence of shear. The maximum compressive stress in a horizontal arch slice of constant thickness is in the intrados at the abutment. Therefore, an arch with lesser thickness at the crown than at the abutment would prove more economical. However, for simplicity of calculations in preliminary designs, it is suggested that horizontal arch slices of constant

thickness be used and the intrados stress p_i at the abutment be used for the criterion of design.

Figure 3 indicates the ratio R of maximum compressive arch stress p_i at the abutment obtained by the elastic method to the arch stress obtained from Eq. 1 by the cylinder method. It will be noted that, for a value of $r/t = 10$ and a central angle 2α of 100 degrees, the arch stress according to the elastic method is 1.85 times that obtained by the cylinder method.

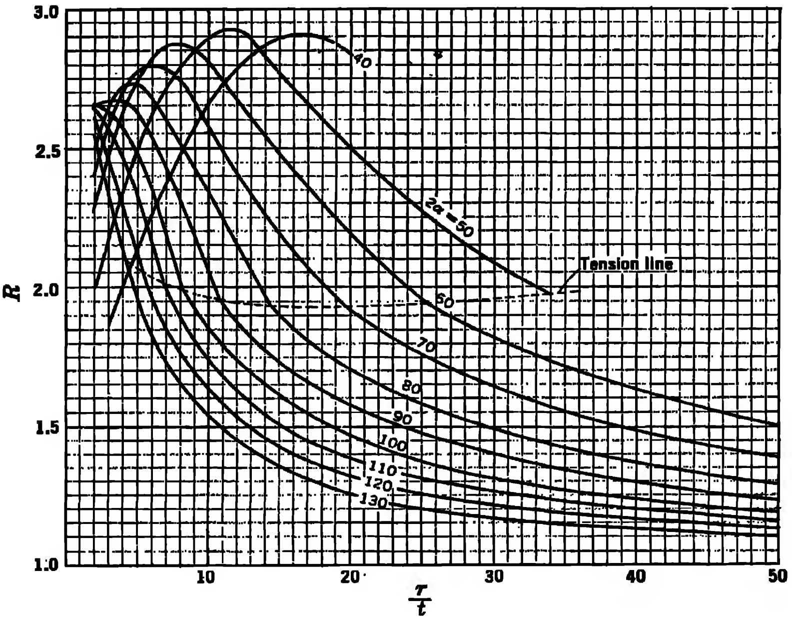


FIG. 3. Coefficient R for arch design at abutment.

Figure 3 was obtained by aid of the plotting of the Cain equations made by F. H. Fowler [3] and adjusted by consideration of the "secondary arch" theory, explained by B. F. Jakobsen [2, p. 1545], to compensate for the assumption of no resistance to tensile stresses. This tension exists for all values of R above the tension line shown in Fig. 3.

The maximum compressive stress in the arch may be obtained by multiplying the stress computed from Eq. 1 by R, or

$$n_i = pR = \frac{w_2 h r_e R}{t} \tag{2}$$

or

$$t = \frac{w_2 h r_e R}{p_i} \tag{3}$$

where R is obtained from Fig. 3.

5. Comparison of Cylinder and Elastic Methods. Dams have been designed by the cylinder method, using a reduction in allowed stresses to account for the lack of consideration of the elastic properties and temperature changes in the concrete, as previously explained. In fact, this method is still advocated by some for low, unimportant dams. However, the procedure is very inconsistent because the required reduction in allowed stresses is not constant at different elevations, being very small for the thin, slender arch slices near the top of the dam and very large for the thick, stiff arch slices at the bottom. In the example of Section 11, an allowed stress of 700 lb per sq in. is used for the elastic method. To get the same theoretical thickness by the cylinder method, a stress of 610 lb per sq in. 20 ft from the top and 180 lb per sq in. at the bottom must be used. Therefore the cylinder method is permissible only for the first approach in field studies where more accurate methods are not conveniently at hand.

6. Trial-load Method. For final designs of important dams, particularly those located on unsymmetrical sites, the trial-load method may be used. This method includes all the conditions known to affect the stresses in the dam.

At any point on the surface of the dam, the horizontal arch deflection must equal exactly the vertical cantilever deflection. If we had only one completely loaded arch ring and one completely loaded cantilever, we could easily determine the division of the total load between the two elements of the structure. However, this relation being obtained for one point, it must agree exactly with other points directly above, below, and on each side. Each of these other four points has, in turn, four other adjacent points to which the deflections must agree. A dam must be laid out by one of the simpler methods and the stresses determined, considering not only the stresses due to the water load but also those due to internal changes as above listed. The solution can be obtained only by trial, assuming various divisions of load between arch and cantilever, and the resultant deflections, until the deflections of each arch ring at all points conform exactly to those of the vertical cantilevers at the same points. The problem is exceedingly complex and will not be included here. It has been used only by government agencies in very important cases.

7. Model Tests. Model testing of arch dams has not been developed to an extent sufficient for use as a substitute for analytical methods. Photoelastic studies are useful chiefly as a check under unusual conditions.

8. The Central Angle. Jorgensen [4] has shown that the theoretically most economical central angle 2α of an arch dam at any elevation, when designed by the cylinder method, is 133° 34′ under any combination of span, loading, and allowed stress. But, for the adaption of the elastic theory herein used, the most economical central angle is somewhat larger. However, the best central angle depends upon the topography at the site, the site shown in Fig. 11 being adaptable to a larger central angle than that of Fig. 5. Very few constant-central-angle arch dams have been built with a central angle exceeding 110 degrees. They average not more than about 100 degrees.

The maximum practical central angle should be adopted. The use of any smaller central angle at any elevation is not consistent with economy. Thus

the constant-angle arch dam is the most economical and has been adopted for most modern dams. The constant-radius dam, which has a maximum central angle at the top, varying to an extremely small central angle at the bottom, contains 50 to 80% more yardage than a dam having a constant angle. However, as explained in Step 12 of Section 11, both the radius and angle may have to be varied to suit local conditions.

9. The Minimum Thickness of Dam. Aside from the thickness of the dam theoretically required to keep the stresses within allowed limits, the minimum permissible practical thickness, dictated by other features, is as follows:

(*a*) George E. Goodall (Consulting Engineer, Sacramento, Calif.) proposes that, to have an ample factor of safety (about 12) against buckling in the upper horizontal arch slices, the thickness of the arch at any elevation should be obtained from the following equation [5], provided the allowed stress, computed as previously described, is not exceeded:

$$t = 2.5r \sqrt[3]{h \div \left[E\left(\frac{17,250}{\alpha^2} - 1\right)\right]} \qquad [4]$$

where E is the modulus of elasticity of concrete in pounds per square inch, which may be taken as 3,500,000 for preliminary studies.

(*b*) The foregoing buckling equation results in zero thickness at the top of the dam. To conform with conservative modern practice, the ratio of length of the arc to the thickness of the top of the dam should not exceed about 60 and at midheight should not exceed about 20.

9A. Temperature Stresses. For preliminary designs it is permissible to consider only the temperature stresses caused by the maximum possible average temperature drop F in the concrete after the vertical construction joints are grouted. If the joints are grouted when the dam is at its mean annual temperature, the temperature drop may be taken from the following empirical equation [11]:

$$F = \frac{340}{t + 8} \qquad [4A]$$

If the dam can be completed before the beginning of winter, left out of service during the cold season, and then the vertical joints grouted before filling the reservoir, the temperature drop will be reduced to a minimum. In such cases one-half the value of F in Eq. 4A may be used. Excessively thick arch dams should be artificially cooled.

Stresses caused by temperature changes vary directly, not only with the temperature change F, but also with the coefficient of thermal expansion and contraction C'_F and the modulus of elasticity E of the concrete. For approximate designs, values of $C'_F = 0.000006$ and $E = 3,300,000$ lb per sq in. may be used.

Figure 3A (Fig. 21, Chapter 13, of *Engineering for Dams* [1]) has been calculated with these values and also on a value of $F = 10$. Thus the value

of FC_tE used in the calculation was equal to 200, for other assumed values, multiply the stresses given in the figure by the ratio of the assumed value of FC_tE to 200. Although Fig. 3A was obtained originally on the assumption that the concrete and abutments will resist tensile stresses, it is probably sufficiently accurate for preliminary designs.

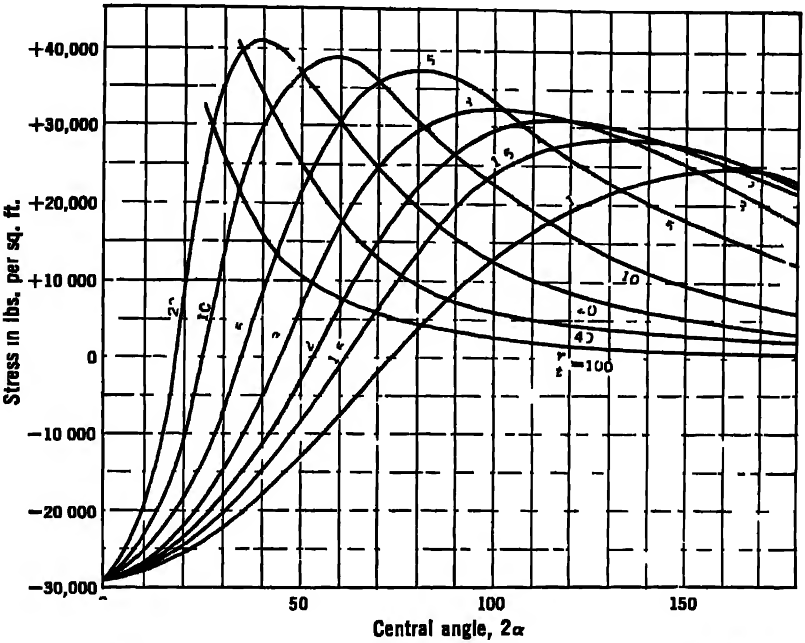


Fig. 3A. Temperature stresses at abutment intrados for FC_tE = 200.

10. Design of Constant-angle Arch Dams. In the design of constant-angle arch dams, each horizontal arch slice is considered separately and its thickness determined from Eq. 3 and Section 9.1. It remains then to place each arch slice in the correct position relative to the arches above and below it. The final arrangement will consist of a downstream overhang at the highest part of the dam and an upstream overhang near the abutments.

A downstream overhang is not objectionable, as the resulting tendency to tip is resisted by arch action. An unsupported upstream overhang is objectionable as it has a tendency to tip away from the arch. A suggested method of accomplishing the best arrangement is given in the next section. Some engineers prefer that the overhang shall not exceed an amount that will cause the resultant of forces at any section, with empty reservoir, to fall within the outside ¼ or ⅓ of the base. Vertical stresses due to overhang in large dams must of course be investigated.

11. Explanatory Example. Figures 5 and 7 represent a contour map of the site, the contours for this hypothetical case being assumed parallel for simplicity. Figure 4 is a cross-section of the site. In an actual design, Fig. 4 is not necessary. If used it should show the chord length measured from the intrados at each elevation instead of a cross-section in a single plane.

It is required to design an arch dam for this site in accordance with the following assumptions:

p' = compressive strength of concrete at 28 days, 3200 lb per sq in.
P = allowed maximum compressive stress for preliminary designs, 22% of 3200 = 700 lb per sq in., or 101,000 lb per sq ft.
w_2 = weight of water, 62.5 lb per cu ft.
2α = central angle, 100 degrees.
E = modulus of elasticity of concrete, 3,330,000 lb per sq in.
F = temperature drop, 10 degrees F.
C_F = coefficient of thermal contraction, 0.000006.

The radius of the intrados of the arch at any elevation is equal to

$$r_i = \frac{l_i}{2\sin\alpha} = \frac{l_i}{1.532} \qquad [5]$$

Step 1. Compute the required thickness and radii of arch at several convenient elevations as shown in Table 1. By the cut-and-try method, a value of t is found for Line 5 such that the value of P in Line 12 will equal 101,000 lb per sq ft.

TABLE 1

COMPUTATIONS FOR THEORETICAL THICKNESS

1. Elevation	100	120	140	160	180	200	220
2. h	120	100	80	60	40	20	0
3. l_i	80.0	106.7	133.3	160.0	186.7	213.3	240.0
4. r_i (Eq. 5)	52.2	69.7	87.0	104.3	121.8	139.0	156.6
5. Assumed t	21.2	21.6	19.2	9.0	4.8	2.0	0
6. $r = r_i + t/2$	62.8	80.5	96.6	108.8	124.2	140.0	156.6
7. $r_e = r_i + t$	73.4	91.3	106.2	113.3	126.6	141.0	156.6
8. r/t	2.96	3.72	5.03	12.10	25.80	70.50	
9. R (Fig. 3)	2.62	2.60	2.47	1.73	1.37	1.10 *	
10. p_1 (Eq. 2)	68,600	68,000	68,000	81,500	89,500	97,000	0
11. p_t (Fig. 3.1)	32,400	33,000	33,000	19,500	10,500	1,000	0
12. $P = p_1 + p_t$	101,000	101,000	101,000	101,000	101,000	101,000	0

* Extrapolated, does not govern.

Step 2. Referring to Fig. 5, draw the arc 1–2 representing the theoretical zero top thickness of the dam, with center at point 3.

Step 3. In order to avoid an excessive upstream overhang near the abutments, assume a downstream overhang at the center of the dam of 7.5 ft which is about 6.3% of the height of the dam. If this overhang is not sufficient, the calculation must be repeated with a greater overhang. Such downstream overhangs up to 16% have been used. Lay the 7.5 ft overhang off as distance 2 4.

Step 4. From Table 1 at Elevation 100, the bottom of the dam, lay off 4–6 equal to r_f and draw intrados arc 4–5.

Step 5. Lay off equal intervals between points 6 and 3 to represent centers of arch for each elevation shown in Table 1.

Step 6. Draw the intrados arcs for each elevation.

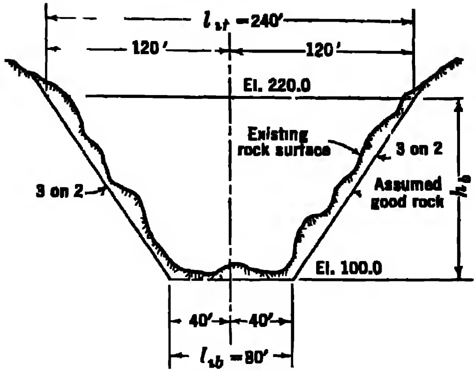


FIG. 4 Cross-section of site for example

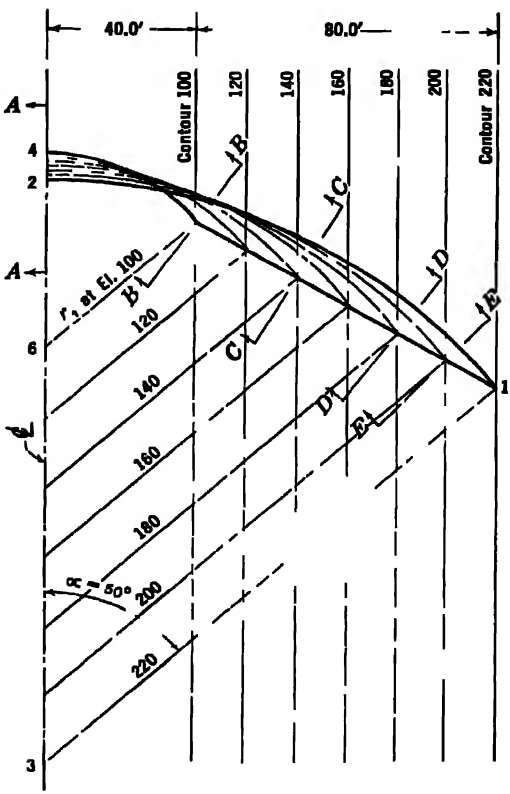


FIG. 5 Lines of intrados for example

Step 7. Take sections as indicated in Fig. 5 and plot the downstream face of the dam as in Fig. 6.

Step 8. On Section *AA* of Fig. 6 plot the required theoretical thickness t governed by allowed stresses from Table 1.

Step 9. From Eq. 4 the required thickness to prevent buckling is

$$t = 2.5r\sqrt[3]{h \div \left[3{,}500{,}000\left(\frac{17{,}250}{50^2} - 1\right)\right]} = \frac{\sqrt[3]{h}\,r}{109.6} \qquad [6]$$

Compute the buckling criterion as shown in Table 2.

TABLE 2

COMPUTATIONS FOR BUCKLING CRITERION

Elevation	100	120	140	160
h	120	100	80	60
$\sqrt[3]{h}$	4.94	4.64	4.31	3.92
r_i (Table 1)	52.2	69.7	87.0	101.3
Assumed t	2.40	3.01	3.50	3.80
$r = r_i + t/2$	53.3	71.2	88.7	106.2
t	2.40	3.01	3.49	3.80

These thicknesses are plotted in Fig. 6 by a dotted line.

Step 10. Plot the necessary thickness for permissible slenderness ratio from Section 9(*b*). At the top of the dam the length of the arc is 273 ft and the permissible thickness is 273/60 = 4.56 ft. For practical reasons, to give ample walkway, the top will be made 6.0 ft. This is plotted on Section *AA* of Fig. 6.

In the same way, the length of the arc at midheight or Elevation 160 is 182 ft and the required thickness at that elevation is 182/20 = 9.1 ft. Since the theoretical thickness is 11.0 ft, no adjustment need be made.

It will be noted that the thickness required to prevent buckling is not a governing condition for this particular example.

Step 11. Plot on Section *AA* of Fig. 6 the enveloping "practical profile."

Step 12. Use the practical profile to plot the other sections in Fig. 6. It will be noticed that the upstream overhang is too great in the vicinity of Section *DD*. There are three remedies for this condition:

(*a*) Increase the downstream batter at Section *AA*.
(*b*) Thicken the lower part of the section as shown.
(*c*) If this required thickening is at an elevation that is below the influence of the "practical profile," increase the radius in the vicinity of that elevation; this will result in a required increased thickness of arch ring.

Step 13. From Figs. 5 and 6 draw the final plan of the dam as shown in Fig. 7.

12. Conditions at the Abutment. An approximate location, d, Fig. 8*a*, of the horizontal component of the resultant force at the abutment, can be obtained for any elevation from Fig. 9, as explained in Table 3.

The direction β, Fig. 8*a*, can be obtained from Fig. 10 as indicated in Table 3. Values of d/t and β from Table 3 are plotted in Fig. 8*a*.

The location of the resultant and the stress at the intrados being known, the amount of the resultant R can easily be determined. If the stress in the

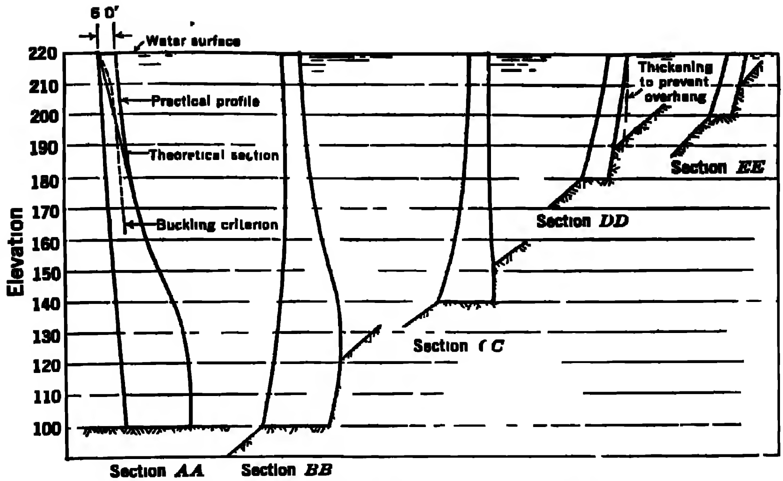


Fig 6. Sections of arch dam for example

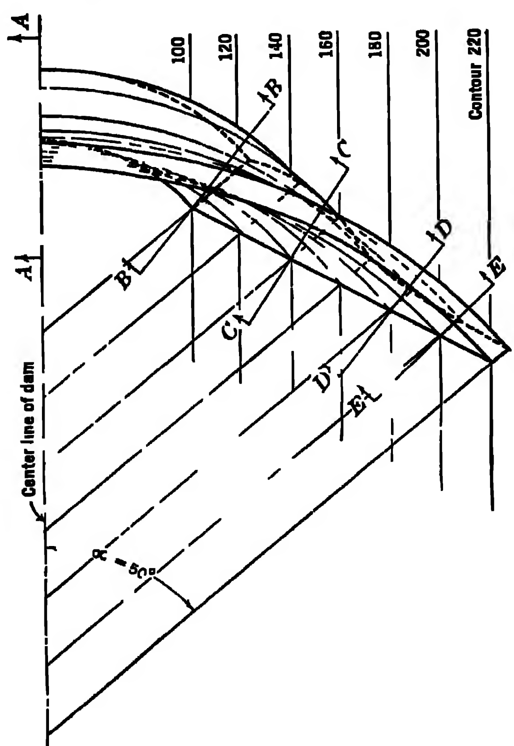
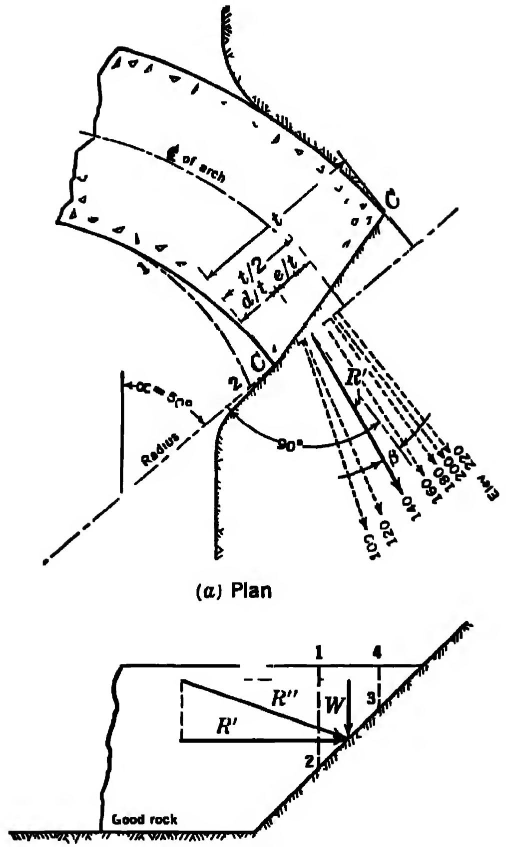


Fig 7 Half-plan of arch dam for example



(a) Plan

(b) Profile

Fig 8. Abutment conditions

TABLE 3

Computations for Location of Resultant Force at Abutments for Example

Elevation	100	120	140	160	180	200	220
t (Practical, Fig 6)	19.7	20.1	17.8	11	8.0	7.0	6.0
r_i (Table 1)	52.2	69.7	87.0	104.3	121.8	139.0	156.6
r (Practical = $r_i + [t/2]$)	62.05	79.75	95.9	109.8	125.8	142.5	159.6
r/t	3.15	3.97	5.38	9.99	15.72	20.36	26.61
d/t (Fig 9)	0.201	0.210	0.260	0.356	0.400	0.420	0.440
$d = (d/t)t$	3.96	4.22	4.63	3.92	3.20	2.94	2.64
β (Fig. 10)	19.6	14.0	8.6	4.0	1.5	1.0	0.5

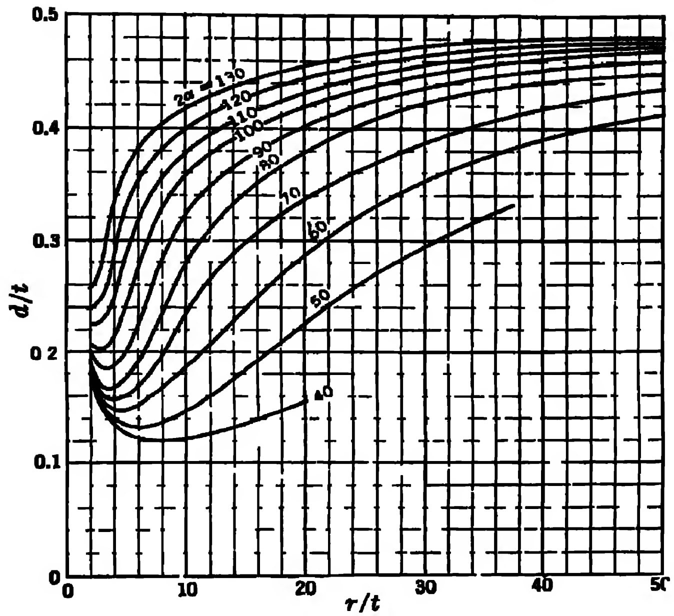


FIG. 9 Location of horizontal resultant force at abutment

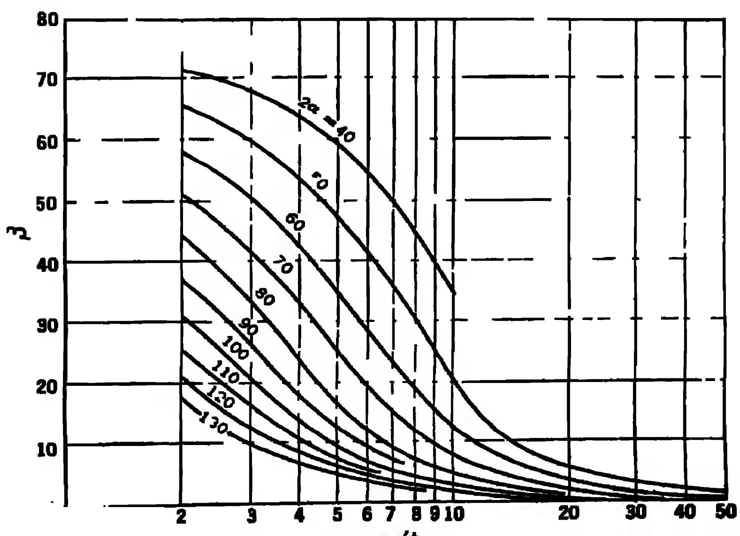


FIG. 10 Direction of resultant force at abutment

abutment is to be reduced, a fillet, shown by line 1–2 in Fig. 8a, can be used and the decrease in the unit pressure determined.

Figure 7 assumes, as is usual, that the abutment excavation is made radial. However, the excavation need be made only to a line such as CC' in Fig. 8a, provided the following criterion is adhered to.

In Fig. 8b let W' represent the total weight of concrete between the vertical radial planes 1–2 and 3–4. Let R' represent the total horizontal thrust (Fig. 8a) between two horizontal planes passing through points 2 and 3. The resultant of W' and R' is represented by R'', which lies in the vertical plane through R' shown in Fig. 8a.

The component of R'' parallel to the plane C–C'–2–3 of the excavation must be resisted by the shear-friction strength of that plane, as indicated in Section 19 of Chapter 17. If the shear-friction strength is insufficient, it will be necessary to step or roughen the foundations.

This criterion must also be applied to any stratifications and joint or fault planes beneath the surface of the excavation that are free to slide.

13. Earthquakes. The effect of earthquakes on arch dams is not well known. The most severe effect on the dam is when the earthquake acts in the direction of the chord of the arch, since it throws an eccentric loading on the arch. The effect of the inertia load of the concrete can be determined, but the nature of the added load due to the inertia of the water is not known. Should an earthquake be expected to occur at the dam site, an allowance for the increased water pressure may be made approximately by assuming radial action and increasing the value of w_2h in Eq. 3 by the unit inertia of the water p_e from Eq. 13a of Chapter 17 plus the inertia of the masonry from Eq. 13 of that chapter, and assuming both to act radially.

14. Spillway Arch Dams. For spillway arch dams, the top is shaped to fit the jet of water as explained in Section 33 of Chapter 17 and the jet is allowed to spill freely. Ample aeration under the jet should be provided, and the foundation should be protected from the falling water. The spillway should be confined to that part of the dam which has a level foundation to avoid a flow of water down the abutment at the base of the dam.

The depth of tailwater required to prevent erosion of the foundation is discussed by Attore Scimemi on p. 1016 of the 1947 *Trans. A.S.C.E.* A sufficient depth is sometimes provided by the use of a small auxiliary dam at any convenient place below the arch dam. Such an auxiliary dam was used at Calderwood. (See Fig. 24, p. 82 of Ref. 1.)

15. Ice Thrust. Arch dams should, of course, be proportioned to resist ice pressure. The ice pressure (Section 8 of Chapter 17) may be considered to act radially and may be taken by a portion of the arch equal in height to twice the thickness of the arch plus the thickness of the ice sheet.

16. Example. Figure 11 shows a layout of the Lake Loveland arch dam. The feature of this dam is the unusual extent of downstream overhang.

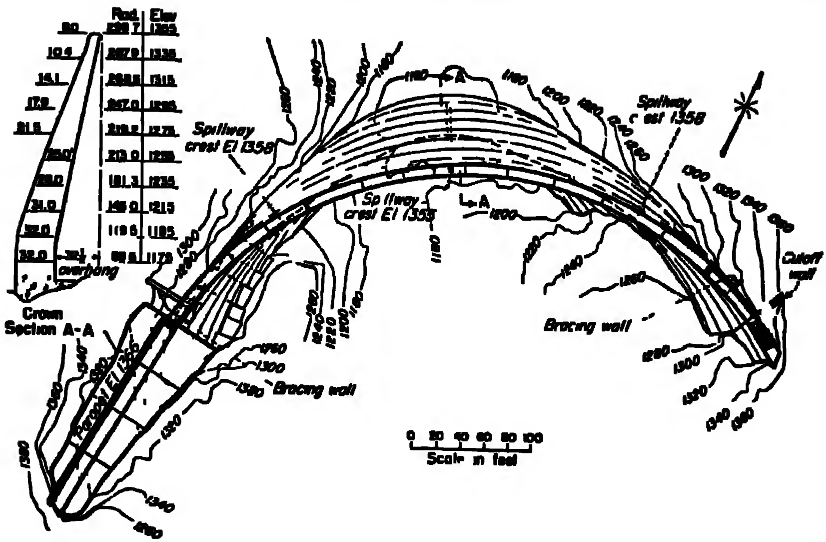


Fig. 11. Lake Loveland arch dam. (G. F. Goodall in *Eng. News-Rec.*, Dec. 26, 1946, p. 76.)

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CHAPTER 19

BUTTRESSED CONCRETE DAMS

(Including Flat-slab, Multiple-arch, and Round-head Buttress Type of Dams)

1. Description. Hollow buttressed dams of many types have been constructed, the usual type consisting of a series of parallel, equidistant concrete buttresses covered by a watertight considerably sloping upstream face and, for spillways, a downstream face and bucket to support the sheet of spilling water, all constructed of reinforced concrete.

Buttressed dams have been constructed on good rock foundations to heights considerably in excess of 100 ft. All buttressed dams on earth foundations should be low and conservatively designed, as unequal settlement is sure to cause stresses for which provision cannot be made in the design.

2. Advantages and Disadvantages. Because of the considerably sloping upstream face, the large vertical component of the water pressure is available to assist stability, so that less weight of concrete is required than for solid dams. In addition, the uplift on the base of buttressed dams on rock is usually less, as described in Section 4.

The lightest type of buttressed dam usually requires only 30 to 40% of the concrete necessary for a solid dam. On the other hand, the buttressed dam requires more cement per cubic yard of concrete, more costly formwork, and more expense in placing concrete, and the dam must be made of reinforced concrete.

More skilled labor is required for constructing buttressed dams than for solid dams. Wages have increased proportionately so much more than the cost of dam building materials that, by 1948, the choice of a buttress dam had lost much of its earlier apparent economy, even where the cost of cement was great and other construction materials were scarce.

There have been many unfortunate experiences with buttressed dams in climates subject to seasons of alternate freezing and thawing. The relatively thin face slabs or arches absorb headwater, and then the concrete sometimes disintegrates rapidly. There are available several means of alleviating this trouble, such as waterproofing the upstream face or using downstream curtain walls or space heating, but nevertheless the hazard from freezing and thawing of the face slabs of buttressed dams is a considerable deterrent to the use of such dams in temperate or frigid climates, or even at

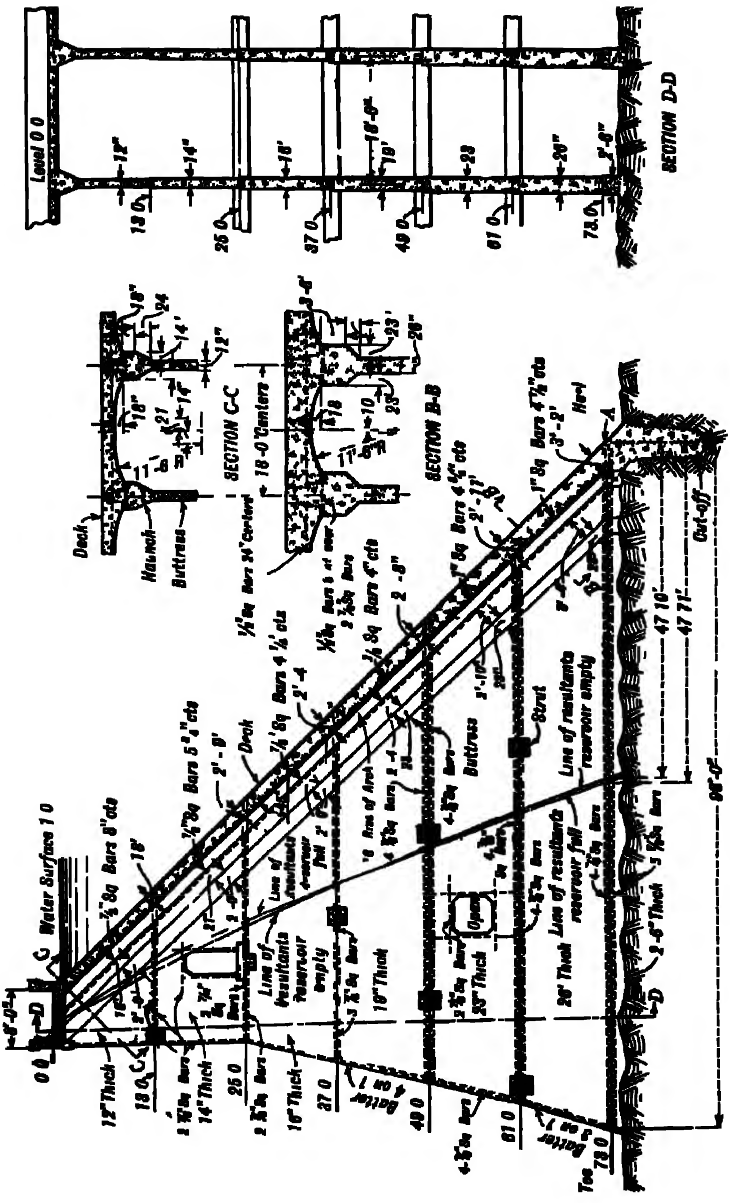


Fig 1 Example of buttressed non-overflow dam.

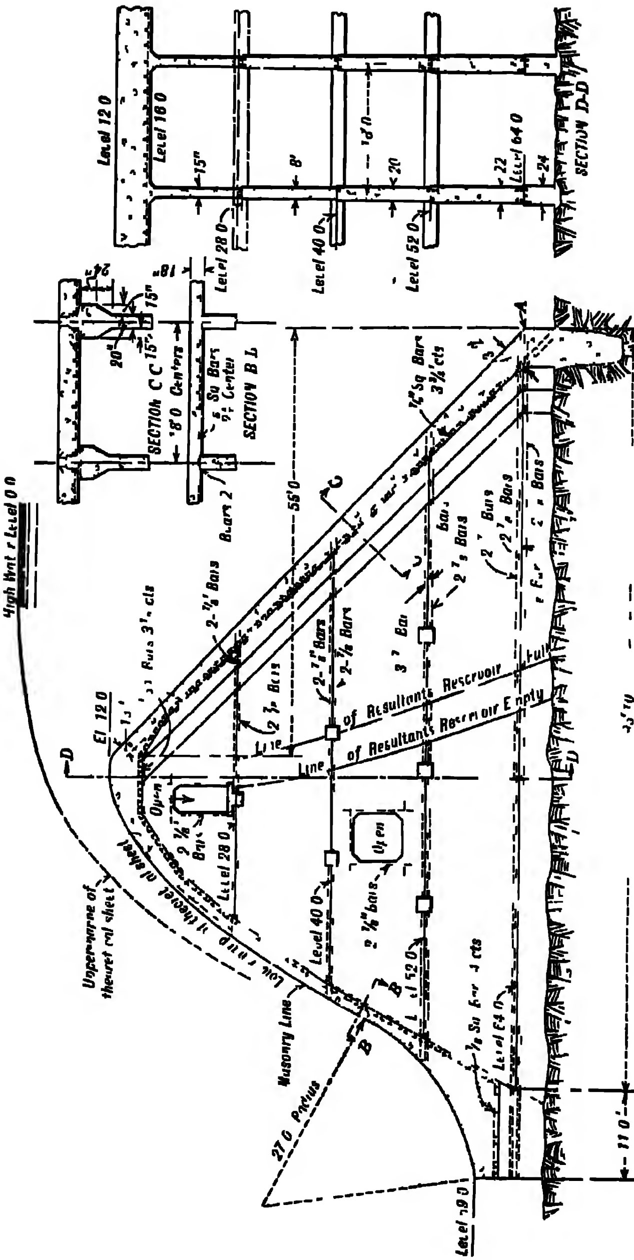


FIG. 2 Example of buttressed spillway dam.

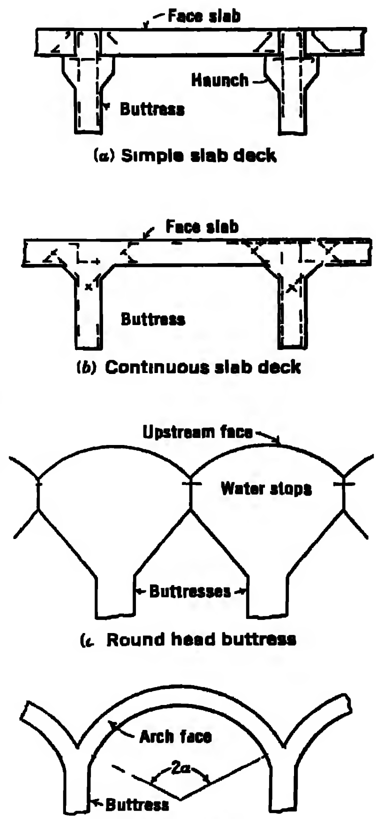
very high elevations in the tropics. This disadvantage does not apply fully to the round-head buttress because of the thickness of its heads or to any buttressed dam whose face slab is perfectly waterproofed if such exists.

3. General Design. Examples of buttressed dams are given below. The general theory of design already stated for solid dams will apply also to buttressed dams. The latter, however, admit of no direct economical methods of design; the shape of the buttresses, type of decking, and other details are worked up in accordance with the judgment of the designer, and the structure is tested for conformity with the rules of design given for solid dams.

4. Uplift. Buttressed dams on earth provided with foundation slabs to reduce unit bearing pressures as in Fig. 5 are subject to exactly the same uplift pressures as solid dams because without a cutoff to impervious material the small length of the path of percolation does not permit drains in the slabs.

When the dam is on rock and does not require spread footings, headwater uplift is usually neglected because of the easy escape of foundation pressures. However, where the rock is liable to uplift pressures on horizontal seams the foundation should be drilled for drainage.

5. Cutoffs and Seepage. If the foundation is porous the upstream cutoff must be carried to an impervious stratum or if this is not possible the foundation slab must be designed for uplift and made of sufficient length to provide the required percolation distance. (See Chapter 25 for treatment of porous foundations.)



(a) Simple slab deck
(b) Continuous slab deck
(c) Round head buttress
(d) Multiple Arch

Fig. 3. Types of buttress dams.

6. The Upstream Deck. The deck or sloping face on the upstream side in many existing buttressed dams has an angle of inclination with the vertical of about 45 degrees but some designers use angles up to 55 degrees or occasionally even larger angles.

Figures 1 and 3a show typical details of a simple flat slab deck most commonly used. The curved underside of the deck, shown in Fig. 2 is seldom used now. The action of the continuous deck shown in Fig. 3b is not adaptable to unequal settlement of foundations and does not provide for

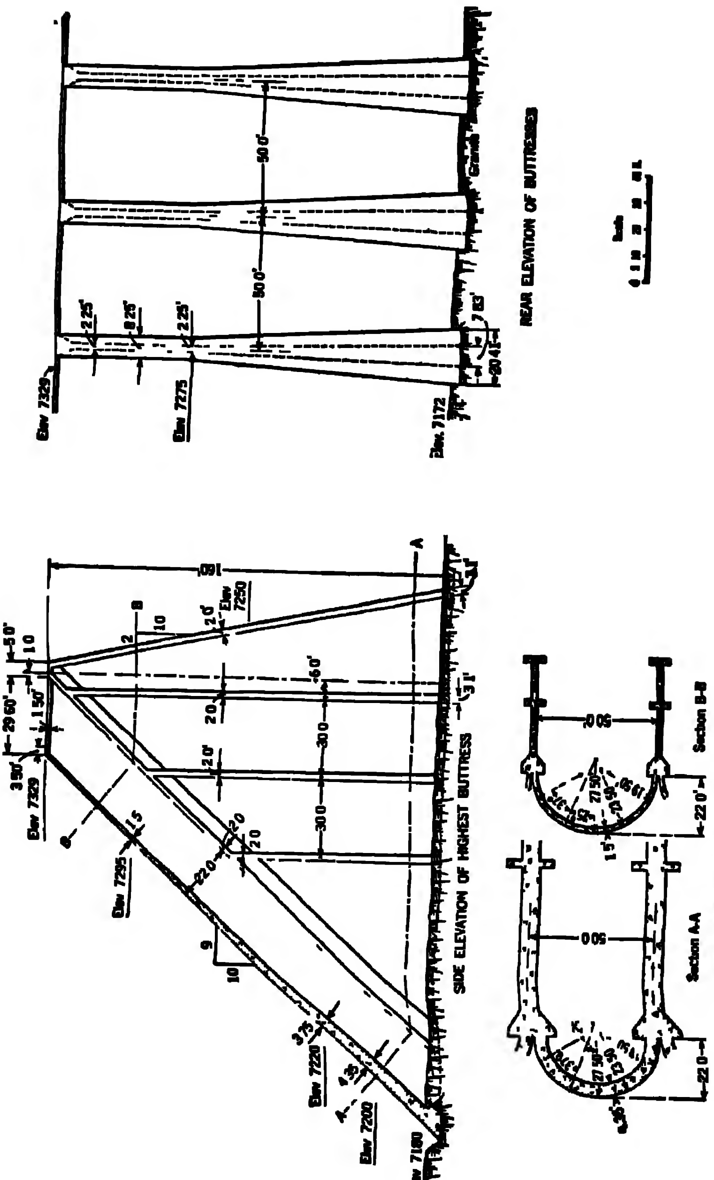


Fig. 4 Florence Lake Dam (*Engineering for Dams* by Hinds, Creager and Justin.)

expansion joints. Figure 3c shows the round-headed buttress type. The buttress heads are enlarged to full span width, and the faces are curved in such a manner that the water pressures are directed radially to the buttresses. The buttress heads are sometimes not reinforced, but an exact analysis will show the necessity for some steel reinforcement at places where tension occurs.

Figure 3d shows a multiple-arch type of face. The water load is taken by a series of inclined cylindrical arches which span the spaces between the buttresses. This deck should be designed by the elastic arch theory, it being remembered that the decks are designed in normal slices and, for each slice, the crown is at a higher elevation than the haunch. Consequently, the loading will be less per square foot at the crown than at the haunch. This is an important consideration, particularly near the top of the dam where the maximum percentage difference occurs.

The central angle 2α has varied from 100 to 180 degrees.

7. The Downstream Apron. For hollow spillway dams of considerable height and those on foundations susceptible to erosion, the dam is provided with a downstream apron, shaped to fit the sheet of spilling water corresponding to the maximum flood to be expected.

The shape of the downstream apron and the bucket should conform to the criteria indicated in Chapter 17 for solid concrete dams. The apron, and particularly the bucket, should be of ample thickness and well reinforced to withstand the impact of ice, trees, and other objects which may pass over the crest. The usual criterion is to use at least the full normal component of the weight of the overflowing jet with allowance for impact on the bucket.

If an apron is provided, provision should be made for thorough ventilation of the interior of the dam by means of openings in the buttresses and an open shaft at each end. The interior of the dam should be well drained to prevent the accumulation of leaks from headwater.

8. The Buttresses. The design of buttresses should follow all the rules of design set forth for solid concrete dams in Part C of Chapter 17. In addition they must conform to the design rules for concrete members, considered as bearing walls, with standard allowed minimum thickness and a standard reduction in allowed unit stress where the unsupported length exceeds a specified amount.

The spacing, thickness, inclination, and other details of the buttresses and deck are adopted tentatively, and the dam is tested for conformity with the rules of design.

Rule 1 may necessitate a change in the length of the buttresses, either by flattening the slope of the downstream end to increase the length of the base or by flattening the slope of the upstream end to increase the vertical component of the water pressure, whichever seems the more expedient.

Rule 2 may require additional weight, which can best be obtained by providing a flatter batter to the upstream face.

Rule 3 may require an increase in area of buttresses by thickening or lengthening or both. When calculating the unit compressive stresses in hori-

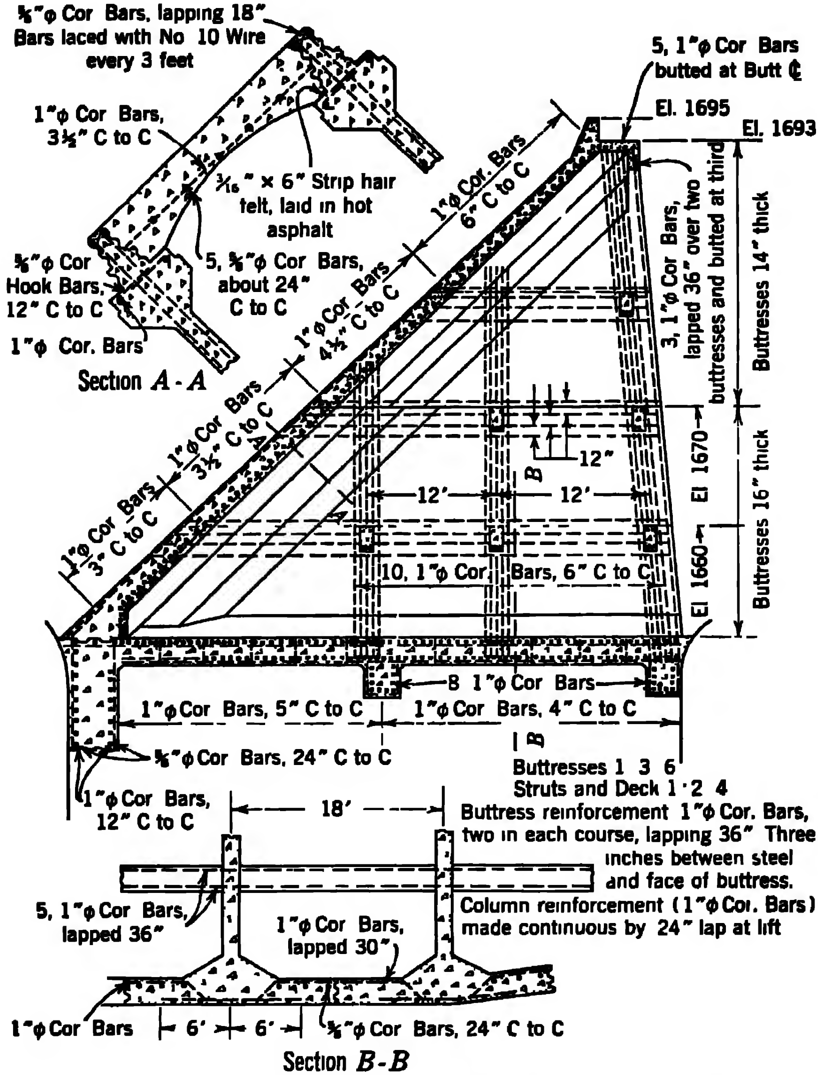


FIG. 5 Details of Mathis Dike Dam (*Eng News*, Vol 74, p 593)

zontal joints and the base, the horizontal area of the deck should be included in the area subjected to stress. This will be on the conservative side for stresses at the downstream face which govern the design. It will not be on the conservative side at the upstream face, but the stresses at that place are

not critical. Rule 4 is not a governing condition in the usual type of hollow dam.

With flat decks, as in Figs. 1 and 3a, the buttresses are spaced from 15- to 25-ft centers. Not many examples of round-head buttresses (Fig. 3c) are

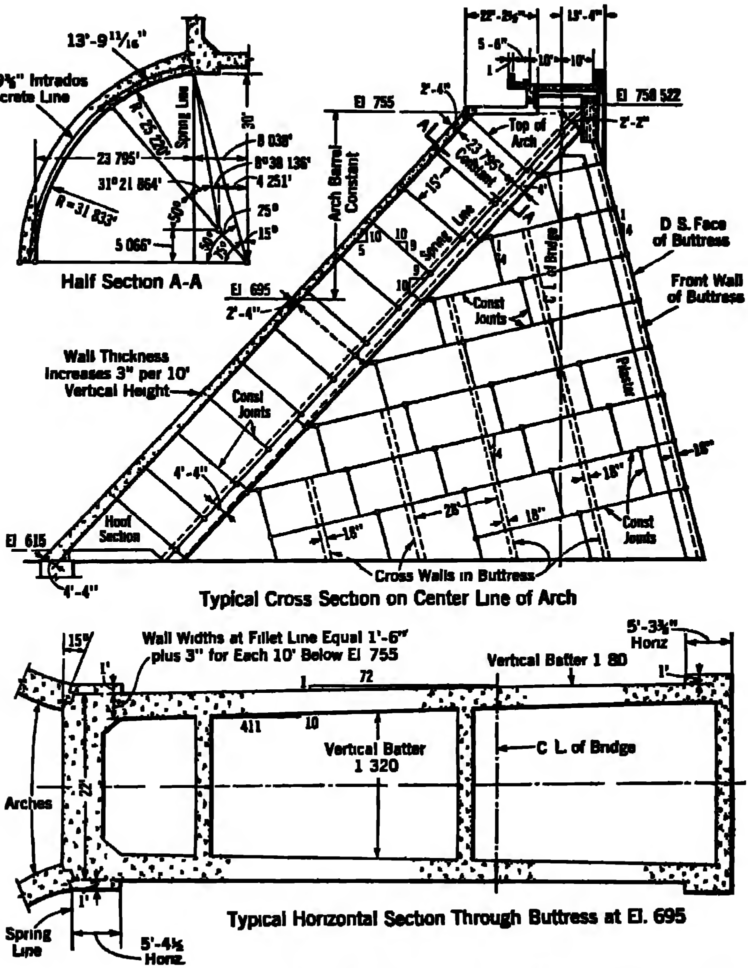


FIG. 6. Pensacola Dam, featuring hollow buttresses (M G Fuller, in *Eng. News-Rec*., Feb. 1, 1940, p. 42)

available The Don Martin dam, of this type, had a spacing of 29.5 ft, with 6 ft 7 in thick buttresses. Multiple-arch buttress spacing (Fig. 3d) is quite variable, ranging, in general, from 30 to 60 ft In general, the most economical spacing of buttresses increases with the height of the dam.

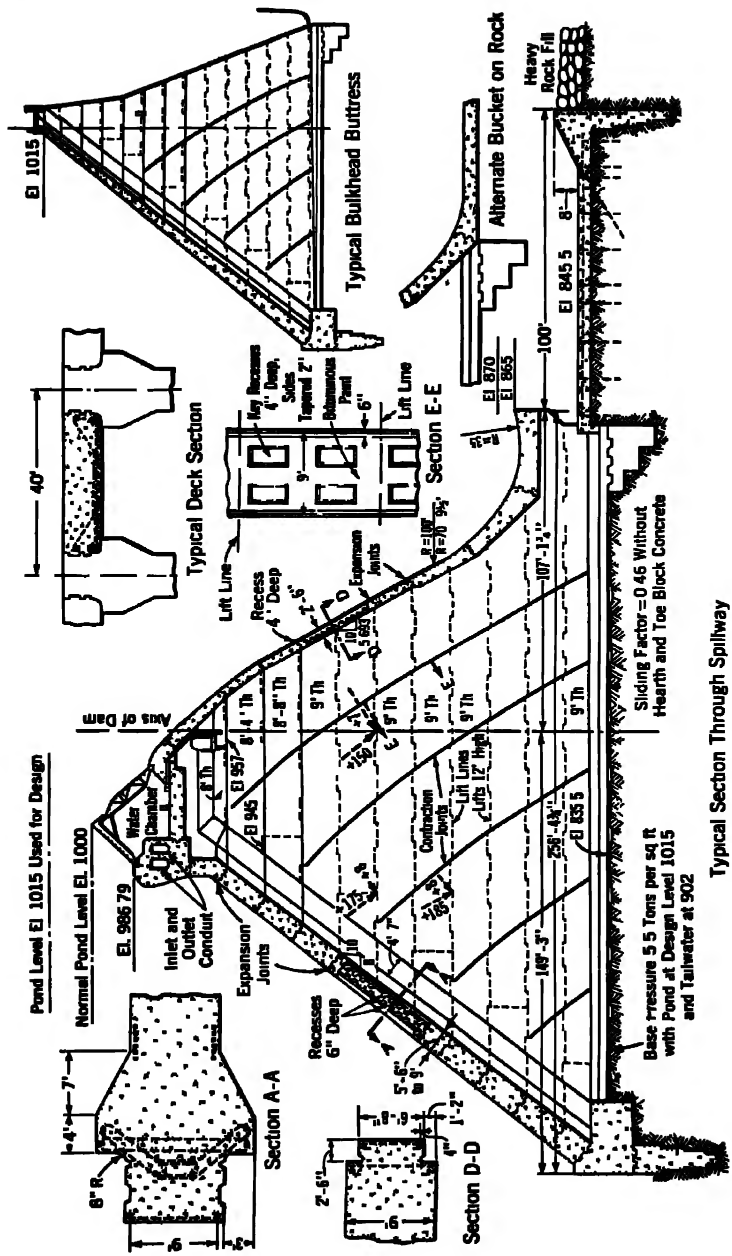


FIG 7. Possum Kingdom Dam, featuring diagonal construction joints (From *Eng News-Rec* June 8 1939 p. 71.)

Where the span of the multiple-arch dam is great, the buttresses are sometimes hollow reinforced concrete as in the Pensacola dam, near Claremore, Okla., indicated in Fig. 6.

Many buttresses have been constructed without buttress reinforcement. Generally, these buttresses have cracked, some so seriously as to require extensive betterments. Others have had reinforcement only at intervals, as in Figs. 1 and 2, and no cracks have appeared. Still others have had shrinkage steel up to 0.3 of 1%.

Shrinkage cracks tend to occur in the buttresses They usually run somewhat in the direction of the planes of minimum principal stress, diagonally from the base towards the deck. Ample standard shrinkage reinforcement should be provided in the buttresses. In the Possum Kingdom dam, contraction joints were provided as indicated in Fig. 7, to influence the direction of the cracks along planes of zero shearing stress.

The buttresses are, in reality, thin walls quite heavily loaded. Where the stresses are high, the unsupported length should not exceed 10 times the thickness. Where the stresses are 50% of allowed stresses, this ratio may be increased to 15.

The unsupported length of buttresses is reduced by the use of struts, as in Figs. 1 and 2, or by pilasters as in Fig. 4.

Open holes in the buttresses are convenient for the passage of men and materials during construction; and an inspection gallery is sometimes provided unless, in low dams, access to the interior may be had from the downstream side at ground level.

The upstream edge of buttresses follows, of course, the slope of the deck. The downstream face, for non-overflow dams, is battered only to conform to designing rules as previously explained. The batter starts either from or slightly below the top of the dam The downstream face for spillway dams conforms to the shape of the spillway apron.

On soft foundations the buttresses may be provided with concrete spread footings. These footings are sometimes of sufficient width to provide a continuous concrete mattress under the dam, as in Fig. 5.

Wind pressure, which is neglected in other dams, may merit consideration if a diagonal wind of high velocity can reach the downstream side. On high thin buttresses, such pressures may increase the danger of buckling. Because the wind cannot strike the buttress face normally, a pressure of 10 lb per sq ft over a width not exceeding the clear distance between buttresses should be safe. Authoritative data are lacking.

9. Allowed Stresses. Stresses and design specifications should conform to the latest issue of the Joint Committee for Reinforced Concrete Design.*

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CHAPTER 20

EARTH DAMS—GENERAL PRINCIPLES OF DESIGN

1. Introduction. The discussion presented in Chapters 20 to 22, inclusive, is confined to a general synopsis of the subject of earth dams and is intended to provide sufficient data for the engineer to prepare preliminary designs and estimates. For a more extensive treatise on the design and construction of earth dams, the reader may consult Refs. 2 and 3 of Section 29 Many of the data given in Chapters 20, 21, and 22 are taken from these references.

2. General. A safe earth dam can be designed and built for almost any given site and foundation condition by utilizing a wide range of earth materials. The authors believe that the construction of any earth dam requires a working knowledge of the science of soil mechanics to determine and analyze foundation conditions and available embankment materials. Dam engineers are rapidly becoming more scientific and less rigidly bound by precedent. The change is all in the right direction as long as there is a proper balance between the practical and the theoretical

Preliminary field investigations are a prerequisite for the design and construction of earth dams The types of investigations and methods of execution are discussed in Chapter 7, "Investigation of Sites"

3. Advantages and Limitations of Earth Dams. When plenty of materials are convenient to the site, earth dams can usually be built at considerably lower cost than any form of concrete gravity dam. The use of this type, however, is often limited by the necessity of providing a more suitable spillway for the passage of floods. It is not safe to allow water to spill directly over the earth dam, even if it is well paved, unless the volume of the flood per linear foot of crest is small. Therefore a spillway of more suitable character is a necessary adjunct. Some spillways would require most, if not all, of the available length of the dam, in which case an earth dam would be out of the question.

The quantity of seepage through pervious material is inversely proportional to the distance the water is required to travel. An earth dam, having the longest base in proportion to the height, is particularly adaptable to sites having pervious foundations.

With proper maintenance, the earth dam should be as permanent as the best. The necessary maintenance charges become rapidly less as the structure settles into its final position and becomes well compacted, tight, and overgrown with proper vegetation to withstand wash from rains.

Earth dams possess a distinct advantage in landscape work where it is desired to change the natural appearance of the site as little as possible.

4. Foundations. Ledge rock foundations for small dams seldom give cause for concern except that in some cases they may require grouting. Foundations of earth dams are often more or less recent alluvial deposits which have not been consolidated under any material load. Coarse sands and gravels in the foundation of an earth dam give no trouble with regard to stability, because, although they may not be consolidated, they will promptly consolidate as the load is applied.

For very fine and uniform sands great caution is necessary. If they are of less than critical density * they may, when saturated under load, flow almost like a liquid if activated by some disturbance as, for instance, an earthquake or even blasting or the passage of trains. Ordinarily any cohesionless material which has dry unit weight of less than 90 lb per cu ft is susceptible. It is possible to consolidate such a foundation to a point where its density is greater than the critical and the foundation is thus no longer subject to flow on disturbance [9].

A plastic clay foundation in an earth dam is usually the type of foundation which requires the greatest amount of study and investigation in order to obtain unquestionable safety. Frequently extremely flat slopes must be used for the earth dam built on such a foundation in order to keep the stresses in the foundation sufficiently less than the strength of the material to provide a suitable factor of safety.

The condition of the foundation is one of the important factors in choosing a dam site. Other things being equal, one would choose the dam site with the best foundation conditions. Over-all economic considerations should govern the choice of site For instance, one might choose a site having a foundation of plastic clay to a considerable depth because, in spite of the flat slopes that would be required, the site would permit a much shorter dam, so that the total cost of the dam at this site would be materially less than that for a dam with smaller cross-section at another location.

5. Materials of Construction. Earth dams have been built successfully of loose rock, gravel, sand of all degrees of fineness, silt, rock flour, and clay. Because of the very much greater quantities of materials involved for an earth dam, the materials must come from the borrow pits and quarries close to the site to avoid prohibitive cost. Of course, if the site were in a rock canyon with no earth at all near by, very little consideration would be given to an earth dam. One would probably build either a concrete or a rock-fill dam.

For earth dams built in rolled layers, materials high in sand and gravel are suitable for the downstream portions of the dam and materials of a fairly impervious nature are desirable for the upstream portions. For hydraulic or

* Critical density of a cohesionless material is that density at which there is no volume change with a further increase in deformation under load.

semihydraulic fill, earth dams, sand and gravel with some rocks and boulders, rock flour, and silt are suitable materials (see Section 11, Chapter 22). Enough fines should be present in the available material to form an impervious core. Though it is desirable to have available ample quantities of both pervious and impervious materials, it is entirely practical to construct a dam almost entirely of either class of materials.

6. Design to Suit Available Materials. In the interest of economy the design of an earth dam should be adapted to the utilization of the materials available at or near the site. Thus, if nothing but sand is available near the site, then the adopted design should utilize this sand for the bulk of the dam, limiting the imported material of concrete, clay, or silt for providing an impervious member to the minimum required.

Some common types of design suitable for the stated materials available and the given foundation conditions are presented in Sections 7 to 11, inclusive, and should not be considered as representing actual designs. Actual designs for a given condition should be arrived at only as the result of field investigation, analysis, and study.

7. Suitable Design with a Sand Gravel Material. In Fig. 1 is shown a suitable design for a site where nothing except sand gravel is available.

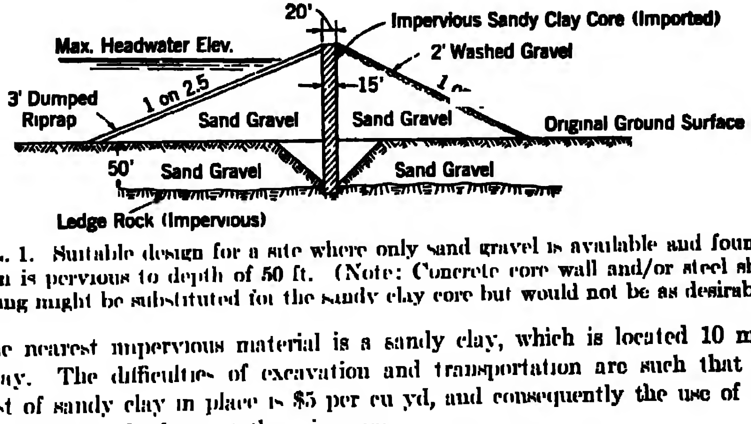

FIG. 1. Suitable design for a site where only sand gravel is available and foundation is pervious to depth of 50 ft. (Note: Concrete core wall and/or steel sheet piling might be substituted for the sandy clay core but would not be as desirable.)

The nearest impervious material is a sandy clay, which is located 10 miles away. The difficulties of excavation and transportation are such that the cost of sandy clay in place is \$5 per cu yd, and consequently the use of this material is to be kept at the minimum.

Accordingly, a trench with 1 on 1 side slopes is excavated to ledge rock a vertical distance of 50 ft. A 15-ft width of the ledge rock is cleaned off very carefully, and a 15-ft wide section of the impervious sandy clay is bonded to it. The rest of the trench is then refilled (in the dry) and thoroughly compacted, using sheep's-foot roller and/or air tampers. For a width of 15 ft in the center of the trench only the impervious sandy clay is used, but outside of that limit the refill is the sand gravel which forms the original foundation.

Above the base of the dam the 15-ft wide core is extended, being placed as a part of the 8-in. horizontal layers in which the dam is being carried up and compacted.

A thin reinforced-concrete core wall 1 to 3 ft thick might be substituted for the sandy clay core if less expensive, but it would not be quite as desirable because the sandy clay core would adjust itself more readily to any slight movement and would also be tighter than the concrete.

The drainage conditions for such a design would be excellent. The cutoff through the foundation would make the downstream portion of the sand gravel forming that foundation available as a drain for the slight amount of seepage water which did get through the core. After passing through the core, the seepage line will drop promptly to the foundation. The upstream slope is somewhat flatter than the downstream slope in order to take care of possible drawdown pressures.

8. Suitable Design with Clayey Silt and Coarse Sand Materials. Figure 2 shows a design suitable for a site where both clayey silt and coarse

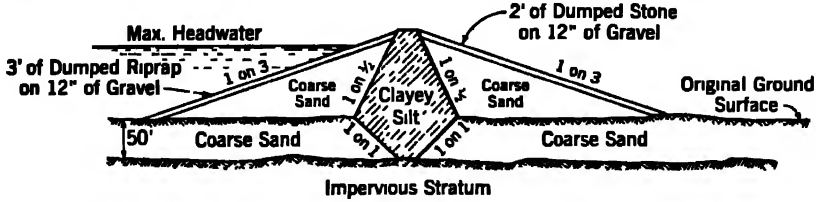

FIG. 2. Suitable design for a site where both clayey silt and coarse sand are available and where foundation is pervious for depth of 50 ft. (Note: Steel sheet piling might be substituted for foundation cutoff.)

pervious sand are available in adequate quantities in borrow pits near the site. As in Fig. 1, both drainage and stability conditions are favorable. The impervious stratum 50 ft below the base of the dam has adequate shear strength. The foundation cutoff leaves the pervious foundation downstream from the cutoff available as a drain for any water that gets through the core or cutoff.

9. Suitable Design with Clayey Silt and Coarse Sand Materials for an Impervious Foundation. The design in Fig. 3 is suitable for a site

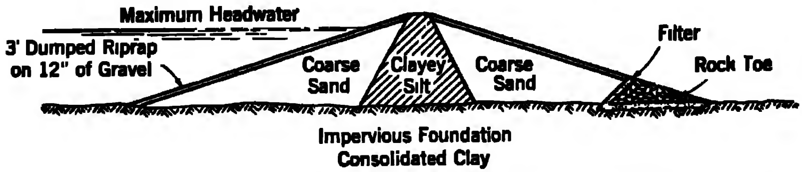

FIG. 3. Suitable design for a site where both clayey silt and coarse sand are available and where the foundation is impervious.

where both clayey silt and coarse sand are available in adequate quantities and where the foundation is impervious (either ledge rock or consolidated clay). In this case it is evident that the water that does get through the relatively impervious central section of the dam must appear at the down-

stream face or toe because it cannot enter the foundation. Hence, to prepare for this condition and to avoid the possibility of sloughing due to saturation, a rock toe with a filter consisting of smaller stones with gravel and sand just ahead of it to protect it against impregnation is utilized. Even without going into the design from a quantitative standpoint, it is evident that seepage will be entirely insignificant

10. Suitable Design with Sand Gravel and Clayey Silt Materials for a Highly Pervious Foundation. In Fig. 4 is shown a suitable design for a site where both sand gravel and clayey silt are available and where the

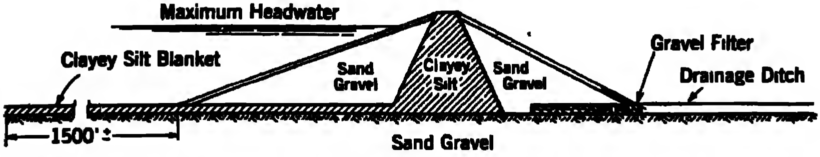

FIG. 4. Suitable design for a site where both sand gravel and clayey silt are available and foundation is highly pervious to a great depth.

foundation is highly pervious to a great depth. A blanket of clayey silt, which is very impervious as compared with the sand gravel of the foundation, is carried from the impervious core upstream under the upstream shell and extended for a distance frequently 10 or more times the head upstream from the upstream toe of the dam. Such blankets cut down the seepage materially by forcing the water to pass through several times the distance which it would have to pass through without the blanket.

Under conditions in Fig 4 the foundation will be full of seeping water, and to take care of seepage a filter layer at the base, which is even more pervious than the sand gravel of the foundation, is provided.

11. Suitable Design with Silty Clay Materials for an Unconsolidated Clay Foundation. In Fig. 5 is shown an earth dam design which is

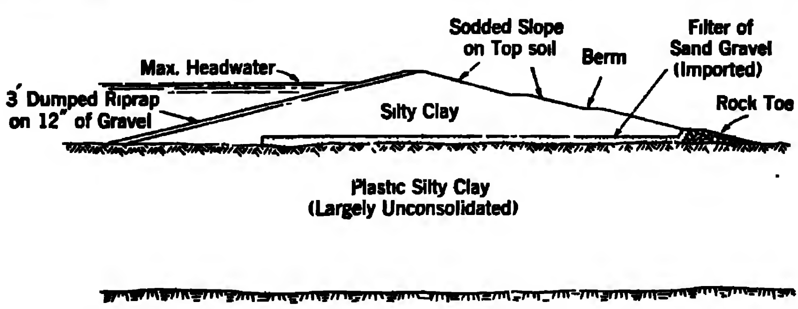

FIG. 5. Suitable design for a site where the only embankment material available is a silty clay and where the foundation consists of a silty clay which is largely unconsolidated.

suitable for a site where the only material available is a silty clay and where the foundation consists of a silty clay which is highly unconsolidated.

In this case the upstream slope is flattened to take care of rapid drawdown, and also in many cases the flatness of both slopes is determined by the requirements for spreading the load so that the maximum unit stress induced in the foundation will be less than the shear strength of the plastic material in the foundation with a fair factor of safety.

It will be noted that a filter layer is placed on the foundation under the base of the dam except near the upstream toe. The filter layer of sand gravel may be "run of bank devoid of clay," as it will surely be vastly more pervious than the silty clay. This filter layer has two particular functions: (1) to provide drainage for the small amount of seepage and thus prevent any possibility of saturation of the downstream face and (2) by providing drainage for water squeezed out of the silty clay by the loading added during construction, it has a marked effect in accelerating consolidation and hence hastens an increase in shear strength of the foundation material.

12. Requirements for the Safety of Earth Dams. The practical criteria for the design of earth dams may be stated briefly as follows: an earth dam should be so designed that

1. There is no danger of overtopping (i.e., sufficient spillway capacity and sufficient freeboard) (Section 13).
2. The seepage line is well within the downstream face (Section 14).
3. The upstream face slope is safe against sudden drawdown (Sections 4 and 6, Chapter 21).
4. The upstream and downstream slopes are flat enough so that, with the materials utilized in the embankment, they will be stable and show a satisfactory factor of safety by recognized methods of analysis (Section 6, Chapter 21).
5. The upstream and downstream slopes of the earth dam are flat enough so that the shear stress induced in the foundation is enough less than the shear strength of the material in the foundation to insure a suitable factor of safety (Sections 7 and 8, Chapter 21).
6. There is no opportunity for the free passage of water from the upstream to the downstream face (Section 28; Section 19, Chapter 22).
7. Water which passes through and under the dam when it reaches the discharge surface has a pressure and velocity so small that it is incapable of moving the material of which the dam or its foundation is composed (Sections 26 and 27).
8. The upstream face is properly protected against wave action and the downstream face is protected against the action of rain (Sections 15, 16, and 17, Chapter 22).

The principal purpose of stating the above criteria of design is to furnish a check list which the engineer can consult to help him make sure that he has considered all the pertinent factors in the design of his dam. An earth dam designed to meet these criteria will prove permanently safe provided proper attention is given to the details of construction.

13. Safety against Overtopping. An earth dam should be designed with the spillway capacity so great that there is no danger of overtopping. A masonry dam with insufficient spillway will generally stand overtopping to a considerable depth without serious damage, but with an earth dam, overtopping usually means failure. Overtopping due to insufficient spillway

capacity is the most frequent cause of failure of earth dams, as shown by a study of more than 100 failures [3]. Many earth dams are in use that have spillways of insufficient capacity to care for floods which are certain to come sooner or later. The subject of spillway requirements is treated in Chapter 6, "Flood Flows."

The freeboard above the greatest expected reservoir surface should be the sum of the heights of tides, seiches, wind setup, and the height to which waves will ride up the upstream face plus a margin of safety based on judgment. Methods for the computation of these heights are given in Sections 10 and 11, Chapter 17.

14. Seepage. Seepage takes place through and under all dams, both earth and concrete. The character of the materials comprising the foundation and the embankment has a very important influence on seepage and its effects. The problem is to minimize and control seepage so that it will have no harmful effects.

In an earth dam the position of the seepage line (defined and discussed in Section 15) influences the final design of the embankment. For a dam of homogeneous material the seepage line will intersect the outside downstream face of the dam some distance above the toe of the dam. A portion of the downstream face becomes saturated and serious sloughing may result, depending on the height of such saturation and the character of the embankment material. It usually is a simple matter to provide drains, discussed in Sections 17, 19, and 21, so that sloughing does not occur. Other methods of lowering or controlling the seepage line are discussed in Sections 22 and 25. A method of predicting the seepage line is given in Section 15.

Once the position of the seepage line (Section 15) is established, the flow net may be readily determined. Using the seepage line as one boundary, the flow net (Section 4e, Chapter 25) is plotted by a cut-and-try process. For the construction of the flow net and the determination of quantity of seepage, see also Ref. 4 of Section 29.

15. Position of the Seepage Line. In an earth dam composed of material so coarse that capillarity has no influence, the "seepage line" is practically the line of saturation, i.e., the uppermost filament of flow. In this case and in all other cases, the seepage line may be defined as the *line above which there is no hydrostatic pressure and below which there is hydrostatic pressure*. This definition must be adhered to strictly whenever the words are used in connection with earth dams composed of any type of material. This is because, where the material is fine enough to be subject to a considerable depth of capillarity, there is saturation without hydrostatic pressure and also a usually negligible flow in the "capillary fringe" above the "seepage line."

The location of the seepage line for an earth dam composed of homogeneous material on an impervious foundation and the point at which it cuts the downstream face is dependent only on the cross-section of the dam. Its position is not influenced by the permeability of the material composing the

dam so long as the material is homogeneous. The seepage line under the assumed conditions has been shown to be fundamentally a parabola [4] with departure therefrom due to local conditions of ingress and egress.

It is always desirable to be able to predict the position of the seepage line in the cross-section of the dam. Following is a rough method of locating the seepage line which is good enough for all practical purposes. The results of this method are in close agreement with the more precise methods and are

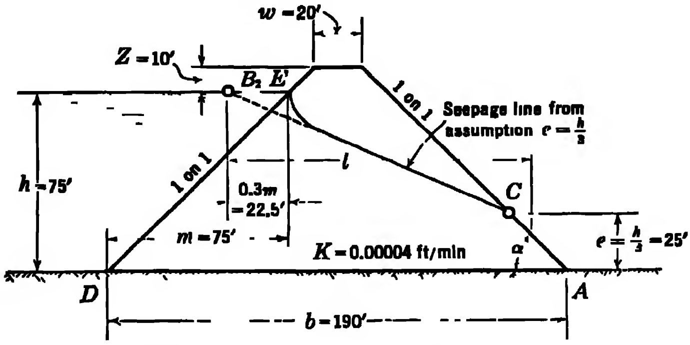

FIG. 6. Determination of the seepage line

more exact than can be obtained by an estimate of the average permeability coefficient for an actual dam.

In Fig. 6

A and D = respectively, the downstream and upstream toes of the dam - if the dam is composed of a relatively impervious core with a pervious shell, A and D, respectively, are the downstream and upstream toes of the core;

B = intersection of the maximum water surface with the upstream slope;

B_2 = intersection of the extended seepage tangent with the maximum water surface;

C = intersection of the seepage tangent with the downstream slope;

m = horizontal projection of the wetted upstream slope;

h = vertical distance from point B_2 to point A equal to the head of water causing seepage;

e = vertical distance from the point A to point C;

l = mean length of seepage path;

b = length of the base of the dam or impervious core, AD.

Referring to Fig. 6, the construction of the seepage line is as follows:

Step 1. If the permeability of the soil deposited in the earth dam is the same in the horizontal direction as in the vertical direction, draw the cross-section of the dam to true scale. If the horizontal permeability is different from the vertical permeability for the embankment material, a transformed

section of the dam must be utilized. To make the transformation, multiply the actual horizontal dimensions by $\sqrt{K_v/K_h}$, where K_v is the permeability coefficient of the material in a vertical direction and K_h is the permeability coefficient of the same material in a horizontal direction. The vertical dimensions remain as they are.

Step 2. Locate point C on the downstream face using $e = h/3$.

Step 3. Locate point B_2 on the maximum water surface at a distance equal to $0.3m$, and draw the seepage tangent (line B_2C).

Step 4. Draw in by eye the ingress transition curve beginning at the intersection of the water surface with the upstream face (point B), perpendicular to the upstream face and coming tangent to line B_2C. Where the upstream face has a very steep slope, the transition may be a reverse curve.

Step 5. Draw in by eye a small egress transition curve tangent to both the line B_2C and the downstream face.

In a composite dam consisting of a highly impervious core with shells of much coarser materials, it is seldom necessary to determine the seepage line for the entire cross-section provided that the position of this line for the impervious core was carefully determined by the method above. In making computations one should bear in mind that, at any given point in the section, the discharge is the same. If the remainder of the top seepage line is desired it may be sketched in by judgment, depending on changes in the permeability coefficients.

If, instead of the impervious foundation assumed in the preceding discussion, a considerable layer of relatively pervious material is the foundation for the dam, for similar conditions and dimensions, the position of the seepage line will in most cases be identical with that for an impervious foundation and the same methods may be used for its location.

The effect of drains and other details of design on the position of the seepage line is discussed in Sections 18 through 25.

16. Quantity of Seepage. For all practical purposes, the quantity of seepage through the dam may be roughly computed, within an accuracy of one's estimate of the permeability coefficient in actual dams, by the following expression based on the construction of Fig. 6 and Section 15:

$$q = \frac{4Kh^2}{9l} \qquad [1]$$

where q = quantity of seepage flow in cubic feet per minute per linear foot of dam;

K = coefficient of permeability in feet per minute;

h = head in feet causing seepage, as indicated in Fig. 6;

l = mean length of seepage path in feet as computed by Eq. 2.

Referring to Fig. 6, the mean length of the path of seepage, l, may be expressed as:

$$l = (1.13h + 2Z) \cot \alpha + w \qquad [2]$$

where α = internal angle formed by the downstream discharge face and the horizontal base;

Z = vertical distance in feet from headwater surface to the top of the dam;

w = top width of the dam in feet.

The discharge through the dam for the example given in Fig. 6 is determined by Eqs. 1 and 2 as follows:

$$l = (1.13 \times 75 + 2 \times 10) \times 1 + 20 = 125 \text{ ft}$$

and

$$q = \frac{4 \times 0.00004 \times 75^2}{9 \times 125} = 0.0008 \text{ cu ft per min per lin ft}$$

This is discharge through the dam only. Discharge through the foundation if pervious may be readily obtained by the proper application of the Darcy formula [4 and 6].

For a section where the horizontal and vertical permeability coefficients differ materially, the value of K used in the above is determined as follows:

$$K = \sqrt{K_v \times K_h} \qquad [3]$$

where K_v is the vertical permeability coefficient and K_h is the horizontal permeability [4].

17. Drainage in Earth Dams. For a dam such as that indicated in Fig. 5, relatively impervious and homogeneous, it is evident that the seepage line would intersect the downstream face well above the toe as in Fig. 6 unless some method of drainage is adopted. In Fig. 5, drainage is accomplished and the seepage line lowered to a point well within the downstream face by the filter layer at the base of the dam.

In some dams drainage is no problem at all because the material in the downstream portion is so pervious that drainage conditions are already excellent. Thus a design like that shown in Fig. 3 with its pervious shells, under the given conditions, provides ample drainage. Even here some attention must be given to the protection of the rock-fill toe. If the rock, say from spalls to several cubic feet in size, were merely dumped at the toe on the consolidated clay of the foundation and not protected at all, in time it would become impregnated with finer material and its efficiency would be seriously impaired or destroyed. Consequently, a filter of graded material (see Section 20) is placed around that portion of the rock toe which would be in contact with the rest of the dam embankment and also with the consolidated clay foundation. This latter point is frequently neglected, but the authors believe that it is frequently advisable to use a horizontal filter layer on top of the impervious foundation because any possible upward movement of water from the foundation might eventually fill the large voids in the rock fill with fine clay or silt and thus ruin the usefulness of the rock toe and cause the seepage

line to come out of the downstream face above the rock toe, with possible sloughing as the result.

18. Effect of Drainage on the Seepage Line. In Fig. 7 it will be noted that the dam is composed of a homogeneous embankment material on a foundation of impervious material but that downstream from the center line there is a drainage layer or filter along the base which discharges through a rock toe fill to the downstream toe of the dam. The upstream end of the filter corresponds to the toe of a dam where α, the internal angle formed by the downstream discharge face and the horizontal base, is 180°.

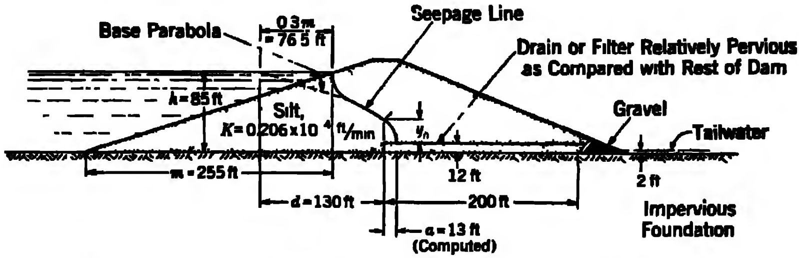

FIG. 7 Effect of drainage on seepage line in homogeneous section as computed in Section 18.

For $\alpha = 180°$, Casagrande [4] gives the following relationships which are written in terms of the nomenclature of Fig. 7:

$$y_0 = \sqrt{h^2 + d^2} - d \qquad [4]$$

$$a = \frac{y_0}{2} \qquad [5]$$

For the example of Fig. 7, $y_0 = \sqrt{85^2 + 130^2} - 130 = 26$ ft and $a = 26/2 = 13$ ft, which means that the seepage line will reach the filter layer at a point 13 ft from its upstream end and a second point on the seepage line measures 26 ft directly above the edge of the filter. Another point is obtained by measuring upstream along maximum water surface a distance equal to $0.3m$ upstream from the point where the maximum water surface intersects the upstream face, m being the horizontal projection of the wetted upstream face.

The seepage line may now be drawn in accordance with the principles described in Section 15. It will be noted that the downstream portion of the dam is practically free from saturation. This is entirely due to the drainage or filter layer which pulls down the seepage line and conducts the seepage to the rock toe. Except for this drainage means, the seepage line would have shown a position similar to that in Fig. 6.

The capacity of the filter should materially exceed the maximum discharge through the dam. By means of Darcy's formula one may check the capacity of the drainage or filter layer. If the capacity of this filter layer is less than

twice the estimated discharge through the dam (by Eq. 1), measures should be taken to increase the capacity by enlargement of filter layer, use of more pervious materials, use of pipe, etc., provided that the drainage layer is not made so coarse that there is danger of the filter's becoming impregnated.

19. Pipe Drains. Pipe drains are sometimes used in earth dams, especially where the material is so extremely pervious that a large quantity of seepage may be expected. Such pipe drains are laid in and are surrounded by a filter of pervious material. Holes are drilled in the pipes of such a size

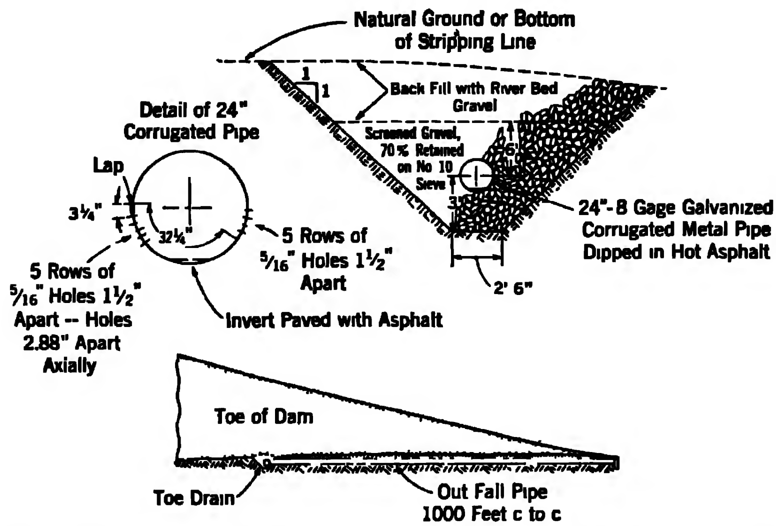

Fig. 8. Galvanized iron asphalt-dipped pipe drain as used at Kingsley Dam, Nebr.

that practically none of the surrounding pervious material can get through them.

Some details of the pipe drain utilized at the Kingsley hydraulic fill dam are shown in Fig. 8; they are typical of several pipe drains which have been used.

One objection to the use of pipe drains is that they may become separated or broken under conditions such that they act as sluice pipes for carrying away large quantities of embankment material in the dam. This objection may be overcome by having the maximum size of filtering material next to the pipe so coarse that it cannot get through the pipe.

Pipe drains are in general used where the rock drains with their filters are not available and where large quantities of water are involved (see also Section 21).

20. Filters for Earth Dams. Filters are often necessary in earth dams to prevent the fine material in the dam from washing out through the much coarser materials usually used for drains and slope protection cover. They should be built up in successive layers of material having a controlled grada-

tion of particle size, ranging from a size equal to or slightly larger than the material in the dam up to a size equal to or slightly larger than the openings in drains or slope protection cover. A number of experimenters, including Stearns [18] and Terzaghi, have determined that a suitable filter layer will adequately prevent the impregnation of a coarse material by a finer material.

The required gradation between successive layers and between the boundary material (as the fill material in the dam) and the adjacent filter layer is controlled by a filter ratio which is defined as the ratio of the 15% size (15% smaller than and 85% larger than) of the coarser layer to the 15% size of the finer layer. G. E. Bertram [5] found this ratio to be 9 or less, applicable

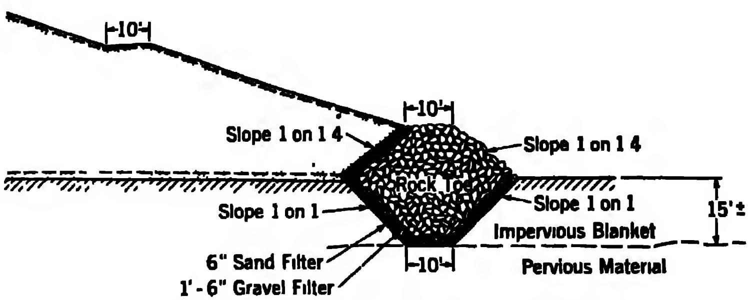

FIG. 9. Typical rock toe with filter protection.

to material at least 50% compacted. If the above condition prevails and if the grain size curves are roughly parallel, practically no impregnation will take place.

The number of layers of filter blanket material will depend on the variation between the particle size in the dam material and the openings or voids in the coarse materials in the drain or riprap cover. In many cases the required results can be obtained with one layer. However, a filter blanket built up of one layer may result in a material with such a wide range of particle size that it will segregate during placing. Nevertheless, filters of a single layer are often the economical way to solve the problem. An 18-in. layer of run-of-bank gravel with 30% of it coarser than ¼ in. and grading up to 2 or 3 in. is practicable. With such a filter there will usually be some impregnation but it will be limited. If more than one layer is required, each layer may range from 6 to 12 in., depending on the particle size and the placing conditions.

Figure 9 shows an actual and rather typical rock toe protected by a filter consisting of a layer of sand and a layer of gravel. There was a particular reason for sinking the rock toe into the original surface as indicated. There was a blanket of impervious material about 15 ft thick which formed the immediate foundation of the dam. Below this impervious stratum there was a deep stratum of coarse sand with some gravel under some hydrostatic pressure. It will be noted that the rock toe drain penetrates to this pervious

material. There was a considerable flow from the pervious stratum through the filter to the rock toe drain which discharged into the old river channel.

21. Position of Filters and Drains. Drains and filters for earth dams may be placed in a number of different positions, each position having both advantages and disadvantages. For usual conditions there is no better drain than a generous rock toe about 25 to 35% of the height of the dam with a proper filter to prevent impregnation by fine materials.

In Fig. 9, a dam with a substantial rock toe is shown. In many locations, however, the rock with which to construct such big rock fills is not available except at prohibitive cost. Consequently, various schemes, including pipe drains (Section 19), are utilized so that proper drainage may be obtained without going to the expense required to obtain a big rock toe fill.

If much hydrostatic pressure is anticipated near the toe, it can be counterbalanced by moving the drain and filter farther upstream to get more weight on top of it, but in that case seepage will be increased owing to the shortening of the path of flow. If the drain is placed at the center line, there is a material increase in seepage, but this may be more than compensated for by the fact that the safety of the structure is greatly improved by better drainage.

Under many conditions the authors favor the use of a filter set back under the dam a distance of 30 to 50% of the distance from the center line of the dam to the downstream toe. If the drain is properly constructed, there is ample safety against piping because of the great weight from the dam. Often the increase in seepage is not enough to be of economic importance.

Figure 8 shows a drain which has been built back from the toe in the cross-section of the dam. If there is any danger of piping at the toe, the drains should be moved back to get greater weight on them.

22. Core Walls and Cutoff Walls. Core walls and cutoff walls of concrete or sheet piling have been extensively used in earth dams to reduce seepage. Cutoff walls are usually differentiated from core walls in that cutoff walls are wholly in the foundation of the dam.

Cores and cutoffs of relatively impervious earth materials (Figs. 1, 2, 3, and 4) are preferably used and are now more frequently used in the dam in place of core walls or cutoff walls to reduce the seepage and lower the seepage line (Section 15). An impervious earth core also has advantages of settling with the dam structure and sealing itself should any cracks develop. However, it sometimes becomes advisable to use core walls or cutoff walls in a location where no relatively impervious material is available but where ledge rock is relatively near the ground surface.

Cores and cutoffs serve to reduce the seepage by increasing the path of flow under the dam, as discussed in Section 4, Chapter 25, and by increasing resistance to flow. Upstream blankets may produce the same effect. However, cutoffs have one extremely important advantage in that a vertical cutoff penetrates a number of strata, thus causing the seepage to pass through the different strata, some of which may be less pervious than the foundation material immediately under the dam. Thus seepage for the same length of seepage

path may be 4 to 50 times less for the vertical cutoff than for the upstream blanket of impervious material. Also cutoffs may penetrate a pervious stratum and seat in an impervious stratum, thus reducing seepage immensely.

Cutoffs are discussed further in Sections 23 and 24.

23. Steel Sheet Piling Cutoff. In many cases steel sheet piling is utilized for foundation cutoffs (Sections 22 and 24), for the reason that, when the depth exceeds about 30 to 50 ft, it is practically always cheaper to drive steel sheet piling than to dig a trench and refill it with compacted impervious soil. At Kingsley Dam, Nebraska, steel sheet piling 125 ft long was driven to connect the core of the hydraulic fill dam to the impervious stratum below (see Fig. 2, Chapter 22). With modern methods of driving and jetting it is feasible to obtain real assurance that the steel sheet piling reaches the recorded depth. By drilling holes each side of the piling it is possible to find out whether or not the piling has curled or gone out at an angle. The kind of steel section chosen is important. If there are many boulders a straight heavy section is desirable, with a strong interlock such as U. S. M 108 weighing 35 lb per sq ft. At Kingsley Dam, however, where there were not many boulders, a section similar to M 112, which weighs only about 23 lb per sq ft, was used. It is not practicable to drive steel sheet piling so that it is absolutely tight. One should never expect it to be as tight as a cutoff trench filled with compacted impervious material.

Jetting is generally essential in reaching material depths with steel sheet piling in sandy and gravelly material. Sometimes, in order to reach depths of 125 to 140 ft, it has been necessary to use jets discharging 200 to 300 gal per min at 300 ft head and in addition to use compressed air with the water. Two to three jets on each side of the piles being driven are sometimes used in a yoke.

24. Partial Cutoff. A partial cutoff is one that extends down from the impervious section of a dam into the underlying strata but does not reach an impervious stratum. In many cases it would be impracticable or extremely expensive to continue the cutoff to an impervious stratum, and, accordingly, the engineer should consider the use of a partial cutoff.

Owing to the fact that alluvial deposits are stratified and that, therefore, the horizontal permeability may be 10 to 50 times the vertical, the effect of such a partial cutoff in reducing seepage may be much greater than might at first appear probable (see Fig. 3 and Section 4, Chapter 25).

25. Upstream Blankets. Instead of a cutoff (Sections 22, 23, and 24) under a dam on a pervious foundation, an impervious upstream blanket may sometimes be used. The purpose of such a blanket is to increase the length of the path of percolation for seepage under the dam and thus decrease the velocity and quantity of seepage (see also Section 4, Chapter 25). A dam with a pervious foundation and a relatively impervious core and upstream blanket is indicated in Fig. 4.

26. Flotation Gradient and Quicksand. Usually quicksand is a relatively uniform fine sand in a rather loose condition completely saturated and

under some slight upward hydrostatic pressure. Actually, quicksand is a condition and not a material. Coarse sands and gravels are sometimes "quick." All that is necessary is that the material should be completely saturated and have enough upward hydrostatic pressure on it to counterbalance the weight of the particles of the material.

If water is passing upward through sand or gravel and the pressure and velocity of the water are gradually increased, a condition is reached where the pressure becomes enough to counterbalance the weight of the particles of sand and gravel in water. At this point there is a sudden loosening and swelling of the material. The sand or gravel is then in what is called a quick condition. It is in an unstable equilibrium, and a slight further increase in pressure will cause the material to flow away like water.

For this critical flotation gradient, Harza [6] gives

$$i_F = \frac{h}{L} = (1 - P)(S - 1) \qquad [6]$$

in which i_F = flotation gradient;
h = difference in head;
L = length of path;
P = porosity or per cent voids expressed as a decimal;
S = specific gravity.

27. Piping. If the head increases slightly over that which gives the flotation gradient of Eq. 6, the material will start flowing, and piping is in progress

Piping occurs when seepage water issues from an embankment or ground surface under sufficient pressure and with sufficient velocity so that the particles comprising the material are carried away.

A number of piping experiments have been made to determine the escape gradient or hydraulic gradient near the point of egress at which piping may start. All the experiments with which the authors are familiar, in general, substantiate Eq. 6.

The possibility of serious piping may be prevented by having the path of percolation sufficiently long in relation to the head, thus reducing the hydraulic gradient (Section 4, Chapter 25), and by providing properly designed and constructed filters and drains so that a dangerous escape gradient will be avoided (Sections 17 and 21).

For safety against piping in dams the minimum ratio of length of path to head should be not less than l/h = 5, which, according to Eq. 6, with specific gravity = 2.65 and per cent of voids = 50% would mean a theoretical factor of safety of 6.

For highly pervious foundations without cutoffs the minimum ratio of l/h should be not less than 8, and 10 is preferable in many cases (see Section 4, Chapter 25).

Under special conditions where drainage and filter systems are specially designed for the given conditions, lower ratios of l/h may be acceptable. A discussion is also given in Section 4*g*, Chapter 25.

28. Cutoff Buttresses on Spillway and Powerhouse Walls. Spillway abutments, powerhouse walls, or other concrete walls extending through the dam in an upstream and downstream direction should be provided with cutoff buttresses projecting well into the embankment. There should be at least two buttresses on each wall passing through the embankment. It is unnecessary to make them more than 12 in. thick if they serve only as cutoffs, but they should be lightly reinforced in order to prevent separation from the main wall. The cutoff walls and the rear face of the main wall should have a slight batter (about 1 on 10), so that shrinkage of the embankment will tend to bring the earth in closer contact with the concrete. This is an important detail since, with vertical walls, there may be a distinct tendency for the earth to shrink away from the concrete.

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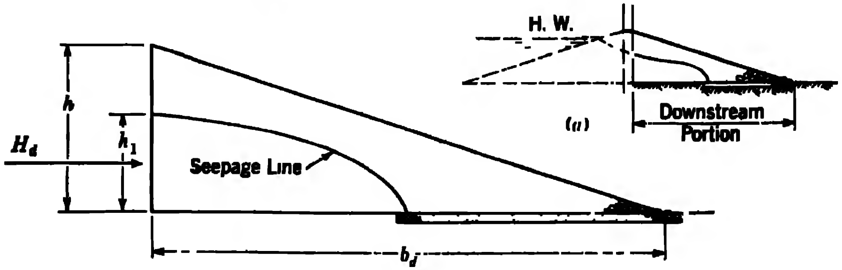
CHAPTER 21

STABILITY OF EARTH DAMS

1. General. Rough and approximate methods presented in this chapter for determining the stability of earth dams are based on the concepts of Coulomb and Rankine, which are already familiar to practically all engineers. Because these methods deal with plane surfaces only and there is generally a curved surface along which resistance to shear may be less, the required factor of safety when using such methods should, in general, be not less than 2. Such methods as presented herein are suitable for use in preliminary investigations. For more precise methods for final analysis of stability the reader may consult Ref. 3 and other references in Section 10.

2. Stability of Earth Dam against Headwater Pressure. If an earth dam meets the criteria of design given in Section 12, Chapter 20, it will generally be found safe against headwater pressure. Usually the headwater pressure in an earth dam is not taken at the upstream face, as it is for concrete dams, but is dissipated in the form of friction throughout the path of flow through the structure, as noted in the concept of the flow net (Section 4e, Chapter 25).

3. Horizontal Shear in Downstream Portion of Dam. The following is a simple method of determining the approximate horizontal shear stress in an earth dam at any given elevation.



(b)

FIG. 1. Shear in downstream portion of dam (see Section 3).

In Fig. 1 a vertical plane is passed through the cross-section of an earth dam; we will consider for the present only the downstream portion of that cross-section.

Let w = the mean unit weight of the embankment material in the central section of the dam. In connection with determining the value of w, take the height of material which is below the seepage line at its submerged (or buoyant) unit weight and the height of material above the seepage line at the center of the dam at its moist or dry unit weight, and then take the weighted mean as w;

w_1 = the equivalent liquid weight per cubic foot of the material of which the dam is composed = $w \tan^2 (45° - \phi/2)$;

w_w = unit weight of water, 62.5 lb per cu ft;

h = vertical distance from top of dam down to base or to the horizontal plane under consideration;

h_1 = vertical distance from seepage line down to base of dam on horizontal plane under consideration;

b_d = horizontal distance from top to bottom of downstream face;

S_d = average unit shear on downstream half of base of dam on horizontal plane through dam per 1 ft width of dam;

S_{md} = maximum unit shear on downstream half of base of dam or on a horizontal plane through dam corresponding to h;

ϕ = angle of internal friction of the material in the dam;

H_d = total horizontal shear on the downstream portion of dam in accordance with Rankine.

$$\frac{w_w h_1^2}{2} = \text{pressure of headwater}$$

$$H_d = \frac{w_1 h^2}{2} + \frac{w_w h_1^2}{2}$$

$$H_d = \frac{h^2 w \tan^2 [45° - (\phi/2)]}{2} + \frac{w_w h_1^2}{2} \qquad [1]$$

$$S_d = \frac{h^2 w \tan^2 [45° - (\phi/2)]}{2b_d} + \frac{w_w h_1^2}{2b_d} \qquad [2]$$

As indicated in Section 9 the maximum unit shear may be twice the average; hence

$$S_{md} = \frac{h^2 w \tan^2 [45° - (\phi/2)]}{b_d} + \frac{w_w h_1^2}{b_d} \qquad [3]$$

Equations 1, 2, and 3 are directly applicable only to the downstream half of an earth dam.

4. Horizontal Shear in Upstream Portion of Dam. The most severe condition to which the upstream portion of the dam can be subjected is to have the water suddenly and instantaneously drawn out of the reservoir. Inasmuch as to have this happen would require the failure of some of the parts of the structure, this criterion is sometimes objected to as far too severe.

However, with very impervious materials the internal pressures that would result in the upstream portion of an earth dam from drawing down the reser-

voir, say, 10 ft in a week would not be greatly different from what they would be if the reservoir was drawn down 10 ft instantaneously.

If the material in the upstream part of the dam, on the other hand, is relatively clean rock or gravel, it may be assumed that it will drain just as quickly as the reservoir can be drawn down.

In general, in the event of sudden drawdown (see Fig. 2), the upstream portion of the dam would still be saturated on its completion, and, accordingly, one should utilize the saturated weight of the material for all material below the maximum seepage line when computing the shear. On the other hand, when it comes to computing the forces resisting shear, the unit weight utilized for all materials below the maximum seepage line will be the submerged (or

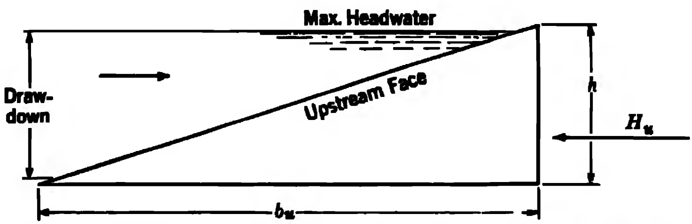


FIG. 2. Shear in upstream portion of dam with sudden drawdown (see Section 4)

buoyant) unit weight of the material except that clean rock and clean coarse gravel may be taken at its dry or moist unit weight both in computing shearing forces and in computing resisting forces.

w_b = submerged unit weight of material in upstream portion of dam.
w_s = saturated unit weight of material in upstream part of dam.
b_u = horizontal distance from top to bottom of upstream face.
H_u = total horizontal shear on upstream portion of dam.
S_u = average unit shear over upstream part of dam.
S_{mu} = maximum unit shear over upstream part of dam.

$$H_u = \frac{h^2 w_s \tan^2 (45° - \phi/2)}{2} + \frac{w_w h_1^2}{2} \qquad [4]$$

$$S_u = \frac{h^2 w_s \tan^2 (45° - \phi/2)}{2b_u} + \frac{w_w h_1^2}{2b_u} \qquad [5]$$

$$\text{maximum unit shear } S_{mu} = 2S_u = \frac{h^2 w_s \tan^2 (45° - \phi/2)}{b_u} + \frac{w_w h_1^2}{b_u} \qquad [6]$$

Equations 4, 5, and 6 are directly applicable only to the upstream half of an earth dam. It should be noted that in this case H_u does not equal H_d (Section 3) because of the concept of sudden drawdown; thus the saturated weight of the material is used. Symbols not defined here are the same as for Eqs. 1, 2, and 3 of Section 3.

The position of the maximum unit shear stress may, without serious error, be taken at a point 40% of the horizontal distance from the top of slope to the point where the slope intersects the horizontal plane or base under consideration.

5. Factor of Safety of Earth Dam against Horizontal Shear. The resisting force against the horizontal shear on the downstream portion of the dam, Fig. 1, is

$$R_d = W'_{ed} \times \tan \phi + cb_d \qquad [7]$$

in which R_d = total force resisting shear in downstream portion of dam, W'_{ed} = total effective weight of downstream portion of dam, c = cohesion or no load shear resistance per unit of area, and the other symbols have the same meaning as in Sections 3 and 4. In computing W_{ed}, the unit weight of the material below the seepage line should be taken at submerged unit weight (w_b) and that above or outside of the seepage line at dry unit weight. Then

$$F_d = \frac{R_d}{H_d} \qquad [8]$$

in which F_d is the average factor of safety against shear in the downstream portion of the dam and H_d is total horizontal shearing force on the downstream portion of the dam as in Section 3. Because the horizontal plane is not usually the weakest plane for failure, Eq. 8 should give a factor of safety of at least 2 in order for the design to be satisfactory.

For the upstream portion of the dam the assumption of sudden drawdown will be utilized as the most severe (see Fig. 2).

$$R_u = W'_{eu} \times \tan \phi + cb_u \qquad [9]$$

in which R_u = total force resisting shear in upstream portion of dam, W_{eu} = total effective weight of upstream portion of dam, and the other symbols have the same meaning as heretofore.

In computing W'_{eu}, the unit weight of the material in the upstream portion of the dam below the seepage line should be taken as the submerged (or buoyant) unit weight except that stone and coarse clean gravel may be taken at unit dry or moist weight. Material always above the seepage line may be taken at dry or moist unit weight. Then

$$F_u = \frac{R_u}{H_u} \qquad [10]$$

In Eq. 10, F_u is the average factor of safety against horizontal shear in the upstream portion of the dam and the other symbols have the same meaning as heretofore.

F_u is also referred to as the factor of safety against sudden drawdown. Because there is generally some other plane which is somewhat weaker than the horizontal, the factor of safety shown by Eq. 10 should be at least 2 for the design to be considered satisfactory.

6. Example 1. Stability of Earth Dam on Impervious Foundation. The determination of the safety of the earth dam on a relatively impervious

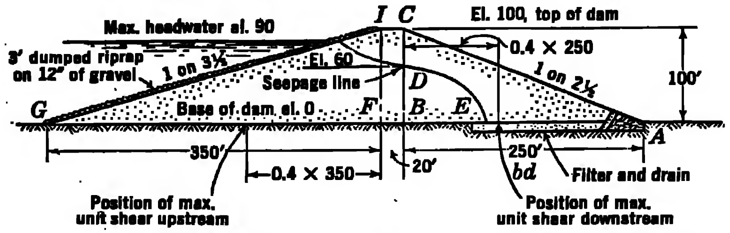


FIG. 3. Assumed design for determining safety factors in horizontal shear.

foundation, indicated in Fig. 3, is presented in Table 1 for the following assumptions:

Dam is of homogeneous material consisting of medium silt placed in 6-in. rolled layers.

Upstream slope is 1 on 3½.

Downstream slope is 1 on 2½.

e = void ratio of material in dam = 0.43 [(Volume of voids)/(Volume of solids)].

n = porosity = $e/(1 + e)$ = 0.30 (percentage of voids expressed as a decimal).

s = specific gravity = 2.65.

w_w = unit weight of water = 62.5 lb per cu ft.

w_d = dry unit weight of material = $62.5 \times 2.65 \times (1 - 0.30)$ = 116 lb per cu ft.

w_s = saturated unit weight = $116 + (0.30 \times 62.5)$ = 134.8 lb per cu ft.

w_b = submerged or buoyant unit weight = $134.8 - 62.5 = 72.3$ lb per cu ft.

w_m = moist unit weight = 120 lb per cu ft.

c = unit cohesion negligible or = 0.

ϕ = angle of internal friction = 32°.

$\tan \phi$ = 0.625.

h = height of dam = 100 ft.

h_1 = height of seepage line at center line of dam = 60.

b_d = length of base in downstream portion = 250 ft.

b_u = length of base in upstream portion = 350 ft.

w = weighted mean unit weight of material

$$\frac{40 \times 120 + 60 \times 72.3}{100} = 91.50 \text{ lb per cu ft.}$$

w_1 = equivalent liquid weight of material in the dam = $w \tan^2 (45° - \phi/2)$.

TABLE 1

Computations for Example 1

Line	Item	Equation	Computation	Result	Remarks
			SAFETY AGAINST DOWNSTREAM HORIZONTAL SHEAR		
1	Headwater pressure		$\frac{W_w h_1^2}{2} = \frac{62.5 \times 60^2}{2}$	= 112,500 lb	
2	$\tan^2 (45° - \phi/2)$		$\tan^2 (45 - 16°) = \tan^2 29°$	= 0.307	
3	H_d	1	$\frac{100^2 \times 91.5 \times \text{line } 2}{2} + \text{line } 1$	= 253,000 lb = 126.5 tons	Total horizontal shear per linear foot of dam
4	W_{sd}	7	$8470 \times 120 + 4030 \times 72.3$	= 1,308,600 lb = 654.3 tons	(See Fig. 3) Area $ABC = \frac{100 \times 250}{2} = 12,500$ sq ft Area $bDI = 60 \times 100 \times 0.67 = 4030$ sq ft Area $ACDb = 8470$ sq ft
5	R_d	7	line 4 × 0.625 + 0	= 409.3 tons	
6	F_d	8	$\frac{\text{line } 5}{\text{line } 3} = \frac{409.3}{126.5}$	= 3.24	This is greater than 2.0 therefore satisfactory (Section 9)
7	S_d	2	$\frac{\text{line } 3}{250} = \frac{126.5}{250}$	= 0.506 ton per sq ft	Average unit shear
8	S_{md}	3	line 7 × 2 = 0.506 × 2	= 1.01 tons per sq ft	Maximum unit shear
9	Unit shear at point of maximum shear		$60 \times 120 \times \tan \phi = 60 \times 120 \times 0.625$	= 4500 lb per sq ft = 2.25 tons per sq ft	See Section 9 and Fig. 3 for location of point of maximum shear
10	Factor of safety at point of maximum shear		$\frac{\text{line } 9}{\text{line } 8} = \frac{2.25}{1.01}$	= 2.23	This is greater than 1.5 therefore satisfactory (see Section 9)
			SAFETY AGAINST SUDDEN DRAWDOWN UPSTREAM HORIZONTAL SHEAR		
11	H_u	4	$\frac{100^2 \times 134.8 \times \text{line } 2}{2} + \text{line } 1$	= 319,600 lb 159.8 tons	Note that complete saturation is assumed whereas the top 10 ft is not actually saturated. This is on the safe side
12	W_{su}	9	$17,500 \times w_f = 17,500 \times 72.3$	1,265,000 lb 632.5 tons	(See Fig. 3) Area $FGI = \frac{350 \times 100}{2} = 17,500$ sq ft
13	R_u	9	line 12 × 0.625 + 0	395.0 tons	
14	F_u	10	$\frac{\text{line } 13}{\text{line } 11} = \frac{395.0}{159.8}$	= 2.47	This is greater than 2.0 therefore satisfactory (Section 9)
15	S_u	5	$\frac{\text{line } 11}{350} = \frac{159.8}{350}$	= 0.457 ton per sq ft	Average unit shear
16	S_{mu}	6	line 15 × 2 = 0.457 × 2	= 0.914 ton per sq ft	Maximum unit shear
17	Unit shear at point of maximum shear		$60 \times 72.3 \times 0.625$	= 2710 lb per sq ft = 1.36 tons per sq ft	See Section 9 and Fig. 3 for location of point of maximum shear
18	Factor of safety at point of maximum shear		$\frac{\text{line } 17}{\text{line } 16} = \frac{1.36}{0.914}$	= 1.49	This is satisfactory (see Section 9)

7. Approximate Shearing Stresses in Foundation. The assumption that an earthen material has an equivalent liquid unit weight which would produce the same shear stress as the material itself has already been used herein The same assumption or theory has been widely applied to the design of retaining walls It will now be applied to determining in an approximate manner the shear stress in a foundation

The criterion for analyzing the stability of the foundation of an earth dam requires that the shear strength of the material in the foundation ($c + wh \tan\phi$) be greater than the shear stresses induced by the weight of the dam The values of the cohesion, c, and the angle of friction, ϕ, are those which apply to the foundation at the state of consolidation that it has at time

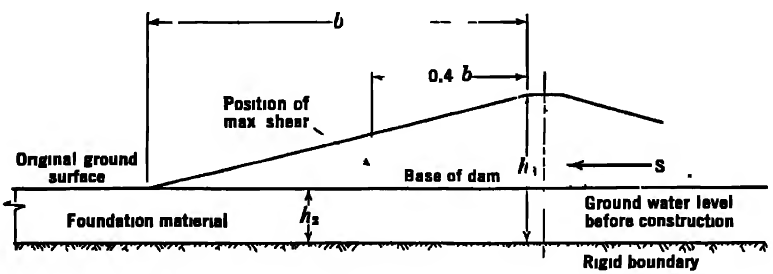


FIG. 4 Approximate shear stress in dam foundation.

of completion of the dam before headwater is raised (see Section 9) The shearing stresses may be determined by the equations given below An example of the determination of the approximate shear stress and the factor of safety is given in Section 8

Figure 4 indicates the one-half section of an earth dam on a foundation the safety of which is to be determined If the earth dam has unsymmetrical slopes then the half section in Fig 4 is that having the steeper slope

$$S = \frac{(h_1^2 - h_2^2)}{2} w \tan^2\left(45° - \frac{\phi_1}{2}\right) \qquad [11]$$

where S = total horizontal shear down to rigid boundary per linear foot of dam,
h_1 = vertical distance from top of dam down to the "rigid boundary," such as ledge rock or a sand gravel stratum the strength of which is great as compared with that of the overlying materials
h_2 = vertical distance from base of dam (or original ground surface), down to the "rigid boundary";
b = horizontal distance along base from top shoulder of slope to the toe of the dam,
w = effective weight per cubic foot of the material in its actual condition

It is here assumed that unit weight of dam and foundation material are the same, if they are different, use a mean (weighted in proportion to depth of each).

ϕ_1 = equivalent angle of internal friction determined as follows:

$$\tan \phi_1 = \frac{c + wh_3 \tan \phi}{wh_3} \qquad [12]$$

in which c is the determined cohesion or no load shear in pounds per square foot and ϕ is angle of internal friction as determined by test.

As S above is the total horizontal foundation shear, the average unit shear is

$$S_a = \frac{S}{b} \qquad [13]$$

in which S_a = average horizontal foundation shear per square foot.

The maximum unit shear may be found from the following relationship, which has been substantially checked by photoelastic analyses:

$$S_{max} = 1.4S_a \qquad [14]$$

in which S_{max} is maximum shear per square foot in the foundation.

Foundations consisting largely of coarse sands and gravels or of thoroughly consolidated silts or clays usually show high shear strength, but foundations consisting of fine, loose, cohesionless materials or of unconsolidated clays and silts may be very defective in shear strength and require thorough investigation.

8. Example 2. Factor of Safety against Foundation Shear. The determination of the approximate shear stress and the factor of safety in the earth dam foundation indicated in Fig. 4 is presented in Table 2 for the following assumptions:

The dam is completed but headwater is not yet raised.

h = height of dam = 100 ft
h_2 = depth of foundation material = 60 ft.
h_3 = vertical distance from top of dam to rigid boundary = 160 ft.
b = horizontal length of base from top shoulder to toe = 400 ft.
w = effective unit weight of material = 120 ft.
ϕ = angle of internal friction = 17°.
$\tan \phi$ = 0.306.
c = unit cohesion = 0.20 ton per sq ft = 400 lb per sq ft.

9. Discussion of Rough Methods of Stability Analysis. If, as the result of the calculations given heretofore, it is found that the over-all factor of safety for either the downstream slope or upstream slope is much less than 2 or if the factor of safety at point of maximum shear is much less than 1.5, then the slopes should be flattened and the stability calculated again. It should be noted that it would be possible to have a factor of safety of less than 1.0 at point of maximum unit shear. In that case there would be local movement at this point, but, if the average factor of safety exceeded unity, the resisting forces would be mobilized and failure would not occur.

TABLE 2

COMPUTATIONS FOR EXAMPLE 2

Line	*Item*	*Equation Number*	*Computation*	*Result*	*Remarks*
1	$\tan \phi_1$	12	$\dfrac{400 + (160 \times 120 \times 0.306)}{160 \times 120}$	$= 0.326$	$\phi_1 = 18°$
2	Equivalent liquid unit weight, w_1		$120 \tan^2 (45° - 18°/2)$	= 63.3 lb per cu ft	$w_1 = w \tan^2 (45° - \phi_1/2)$
3	S	11	$\dfrac{(160^2 - 60^2)}{2} \times$ line 2	= 698,500 lb = 349.3 tons	Total shear in foundation along the length b (Fig. 4)
4	Sa	13	$\dfrac{\text{line 3}}{400} = \dfrac{349.3}{400}$	= 0.873 ton per sq ft	Average unit shear
5	S_{max}	14	1.4(line 4) = 1.4×0.873	= 1.22 tons per sq ft	Maximum unit shear
6	Submerged unit weight		$120 - 62.5$	= 57.5 lb per sq ft	
7	Unit shear below toe		$400 + (60 \times 57.5 \times 0.306)$	= 1,455 lb per sq ft = 0.73 ton per sq ft	Ground-water level is assumed at ground surface
8	Effective unit weight, w		$\dfrac{60 \times 57.5 + 100 \times 120}{160}$	= 96.5 lb per cu ft	At point in foundation under upper shoulder of slope
9	Unit shear at same point as line 8		$400 + (160 \times 96.5 \times 0.306)$	= 5,125 lb per sq ft = 2.56 tons per sq ft	
10	Average unit shear		$\dfrac{(\text{line 7}) + (\text{line 9})}{2} = \dfrac{0.73 + 2.56}{2}$	= 1.65 tons per sq ft	
11	Safety factor		$\dfrac{\text{line 10}}{\text{line 4}} = \dfrac{1.65}{0.873}$	= 1.9	Over-all factor of safety against foundation shear is satisfactory (Section 9)
12	Effective unit weight at point of maximum shear		$\dfrac{57.5 + 120}{2}$	88.8 lb per cu ft	At point of maximum shear in this case there are 60 ft of material below and 60 ft above ground-water level
13	Unit shear at point of maximum shear		$400 + (120 \times 88.8 \times 0.306)$	3,660 lb 1.83 tons	
14	Factor of safety		$\dfrac{\text{line 13}}{\text{line 5}} = \dfrac{1.83}{1.22}$	= 1.5	Factor of safety at point of maximum shear is satisfactory (Section 9)

In Example 1, Section 6, the cohesion was assumed as negligible, but if cohesion is to be taken into account, as it should be if it is significant, a slightly modified value of ϕ may be used in the expression for the "equivalent" liquid weight $w_1 = w \tan^2 [45° - (\phi/2)]$. This modified value of ϕ may be obtained by using Eq. 12.

The position of the horizontal maximum unit shear is empirical, but it is in substantial accord with photoelastic model studies when taken at a point $0.4b$ from the upper shoulder of the slope. The assumption that the maximum unit shear is twice the average is conservative, as photoelastic studies indicate that this ratio is frequently 1.4.

The computations given in Example 2, Section 8, apply to conditions with the dam just completed but the headwater not raised. When the headwater is raised, a new condition will exist which also must usually be investigated. Thus, for the downstream portion of dam and foundation where shear strength equals $c + w_1 h \tan \phi$, all material below seepage line should be taken at submerged unit weight. Also, when investigating safety of upstream portion of dam after raising headwater, the assumption of sudden drawdown should be used. Utilizing the principles of Sections 3 to 8 inclusive, the safety of the foundation with headwater raised may be readily determined.

In many cases the most critical time for the safety of the foundation is when the dam is just completed, as in Section 8. The reason is that, by the time headwater can be raised and sudden drawdown takes place, the foundation will have received a material additional amount of consolidation, resulting in higher values ϕ and c.

The foregoing rough method of analyzing stability of earth dams is simple and quick and gives results which agree quite well with results obtained by other suitable methods. This method may be applied to any horizontal plane through the dam.

10. Stability of Hydraulic Fill Dams. Owing to the process of construction peculiar to hydraulic fill dams (Sections 9 and 10, Chapter 22), the coarse material is segregated and deposited in the shells and the fine materials (clays and silts) are deposited in the core near the center of the dam. The materials in the core, being much finer than those in the shell, stay in suspension longer, and the mixture of water and core material has, when first delivered, a greater unit weight than water. Thus the core when first placed and generally for a long time thereafter is in a more or less liquid condition and exerts a substantial pressure on the shells of the dam, tending to force them outward away from the core. Consequently, a hydraulic fill dam is less stable during construction than at any other time. The desirability of narrow cores is discussed in Section 12, Chapter 22. A typical hydraulic fill dam is indicated in Fig. 2, Chapter 22.

The pressure exerted by the core on the shells may be conceived as being equivalent to that which would be exerted by a liquid having a certain weight per cubic foot. This concept has already been used in the preceding sections Accordingly, the equivalent liquid weight per cubic foot equals $w \tan^2 [45 - (\phi/2)]$, where w = unit weight of the material.

Gilboy [8] presents the following formula for the stability of hydraulic fill dams on the assumption of a fully liquid core.

$$\sqrt{R} = \frac{(C - A)\sqrt{1 + B^2} + \sqrt{C - A}\sqrt{C - B}\sqrt{1 + A^2}}{(1 + C^2) - (C - A)(C - B)} \qquad [15]$$

in which A = cotangent of angle of core slope with horizontal;

B = cotangent of angle of internal friction of shell material;

C = cotangent of angle of outer slope with horizontal;

R = ratio of saturated unit weight of core to unit weight of shell.

Manifestly the factor of safety for the above formula is represented by the degree to which the actual core departs from a condition of liquidity.

In order that the Gilboy method may give directly a factor of safety against failure, an additional ratio will be introduced.

R_1 = ratio of equivalent unit liquid weight of core material to the effective unit weight of shell material.*

$$R_1 = \frac{w_1 \tan^2 (45° - \phi_1/2)}{w} \qquad [16]$$

in which w_1 = unit weight of core material in actual wet or moist condition;
ϕ_1 = angle of internal friction of core, as determined by Eq. 12;
w = effective unit weight of shell material;
R = ratio as previously defined;
$F_s = R/R_1$, in which F_s is factor of safety of the shells against shear from the internal pressure of the core.

11. Bibliography. (See also bibliographies for Chapters 20 and 22.)

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* Thus, in a computation for stability during construction, the unit weight of shell utilized might be the weighted mean between the submerged unit weight and the dry or moist unit weight.

CHAPTER 22

DETAILS OF EARTH DAMS

1. General. The details of design and construction of earth dams deserve special treatment and are adequately discussed in books and papers referred to in the bibliography. The intent of this chapter is to cover those features of earth dams which affect the general design and the choice of method of construction.

2. Methods of Construction. The usual methods of construction are:

(*a*) *Construction in Rolled Layers.* The material is excavated by power shovel, scrapers, drag line, or elevating graders, hauled onto the dam, deposited, spread, moistened, and rolled (see Sections 6, 7, and 8).

(*b*) Hydraulic Fill Method. The material may be excavated by several convenient methods, but it is transported and deposited by the agency of water (see Sections 9 and 10).

(*c*) *Semihydraulic Fill Method.* The material is dumped near the upstream and downstream face of the dam to form rough levees. The space between these levees is then filled with water, and the material placed in or upon the levee is washed toward the center of the dam (see Section 9).

3. Clearing and Stripping. About the first step that must be taken after the erection of the camp and plant buildings is the clearing and stripping of the site. This work should, in fact, be done while the plant is being moved in and erected. Trees and stumps should be removed from the site. The requirements for stripping should vary with the existing conditions at the site and the requirements of the design. For instance, the original surface might be an unconsolidated fat clay which it might be advisable to remove because of its low shear resistance. As a general rule stripping involves only removing the soil (and not necessarily all the roots) and obtaining a bond between the embankment material and the foundation materials.

Some specifications require that all soil containing more than 6% vegetable matter shall be removed from the site; other specifications limit the amount of vegetable matter permissible either in foundation or embankment to 3%.

4. Bonding Dam to Foundation. It is a matter of prime importance to make sure that there is no definite dividing plane between the foundation material and the material composing the embankment. The original surface should be plowed up and dragged with a disk harrow just before any material is deposited on it. Unless the embankment is to be placed hydraulically, or the foundation surface already contains the optimum amount of moisture, the surface should be liberally moistened before the first layer is placed so that

when the roller goes over this first layer it will force the new material down into the old and leave no dividing plane.

5. Puddling. The puddling of clayish soils in the cutoff trenches under dams is of sufficiently common occurrence to require some discussion here. This generally means that the trench is filled with water, and the dry material is then thrown in. It is a nefarious procedure to puddle any material which is high in clay. Clay will sometimes take up 2½ times its own weight of water, when it becomes a slimy mass exerting substantial hydrostatic pressure. Such clay puddled in the core of a dam requires many years to attain a really stable condition. Also in drying it contracts and may leave cracks which produce roofing of the impervious overlying embankment section. A passageway through the impervious section is thus provided.

There is nothing better for a cutoff than a puddle trench refilled with genuine puddle. True puddle is an intimate mixture of stiff clay, sand, and gravel thoroughly tamped into place. Properly constructed, it is superior to concrete as a cutoff, as the material itself is less pervious and also cracks through such puddle walls are practically unknown.

Cutoff trenches are frequently excavated to reach some layer which is more impervious than that immediately underlying the dam. Precautions should be taken to secure a bond between the material refilled into the trench and the material composing the layer which the cutoff trench reaches.

6. Building Embankment in Layers. Unless the material is to be placed and settled by the action of water, it should be spread in thin layers

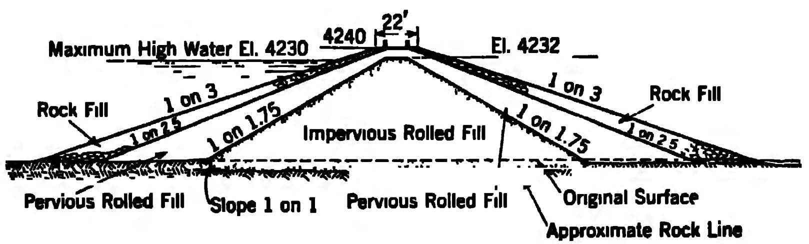

FIG. 1. North and South Dikes, Conchas Dam, New Mexico. (From "Design and Construction of Conchas Dam," U. S. Engineer Office, Caddoa, Colo.)

and rolled. Occasionally dams have been built by dumping the material from trestles much in the manner that railroad fills are made. Such a method should never be used, as it gives porous and unstable embankments which invite piping and sliding.

The layers should seldom be over 12 in. in thickness after rolling and usually should be very much thinner. Probably the best practice with the usual run of pervious materials is to require 8- to 10-in. layers, whereas cohesive materials that are readily compressible should be placed in layers 4 to 6 in. thick.

The desired density having been decided on, field experimentation on the thickness of layers, the amount of rolling, and the weight of the rollers is the

only method of determining the economic manner of obtaining this density. These experiments should be coordinated with the control compaction tests with regard to moisture content.

A typical rolled fill earth dam is indicated in Fig. 1.

7. Wetting Embankment. Impervious cohesive materials when placed should generally have as close to optimum moisture content * as practicable. Excessive "weaving" of the embankment under the moving load of rollers and trucks indicates to experienced engineers and inspectors that the material is too wet. If moisture content is excessive the material should be allowed to lie until the excess moisture has evaporated or drained out before the layer is rolled. It may be advisable to adopt drainage measures in the borrow pit, and sometimes it may be necessary to move to another borrow pit which does not contain an excess of moisture.

The lower portions of high earth dams should be compacted a little on the dry side of optimum moisture to avoid excessive pore pressures. In fact in most cases it is better to place all the material a little on the dry side.

If tests indicate that additional moisture is desirable in the material, it is well to add at least a part of this by sprinkling the previous layer after it has been thoroughly compacted and just before the additional material is added, because then the pressure and kneading action of the roller will force the water through the material in a fairly even manner. The remainder of the water is added to the new layer just before rolling.

With very pervious materials it is usually almost impossible to add too much water. To get the best results from rolling, it is necessary practically to saturate the pervious layer immediately in advance of rolling.

8. Rolling Embankment. The sheep's-foot roller is, in general, the best tool for rolling the embankment, although smooth rollers may be useful for following the sheep's-foot rollers to smooth up the embankment so that it will more readily shed the water in case of rain. The thickness of layers after rolling is generally 4 to 6 in. for impervious material and 8 in. or more for pervious material. The sheep's-foot roller may be arranged to give the desired unit pressure throughout a wide range up to 500 lb per sq in. (assuming one row of feet in contact). The variation in pressure is accomplished by adding or taking off feet and by loading the drum of the roller with water or sand or both.

The unit weight of embankment desired is known in advance and will determine the desirable pressure to use on the sheep's-foot and also the number of passes of the roller. The amount of rolling to obtain the desired unit weight and the necessary moisture content is determined by experiment. Six to eight passes of the sheep's-foot are usual. In general the desired unit weight of the embankment is somewhat greater than the unit weight which would be eventually obtained by natural consolidation due to the pressure of the portion of the dam above.

* The water content at which maximum density is produced in a soil by a specific amount of compaction.

9. Hydraulic and Semihydraulic Fill Dams. *Hydraulic fill* and *semihydraulic fill dams* must fulfill the same criteria of design as earth dams, which are built by depositing material in layers and rolling, but, by reason of the methods of construction used, they offer an entirely different construction problem (see Section 2 for definitions).

The semihydraulic fill method of construction has sometimes been the cheapest and most convenient for a given site. However, certain inherent dangers in this method of construction should be appreciated and guarded against if the method is adopted. Dams built by the hydraulic fill method have been comparatively free from slides during or immediately after construction,* whereas several dams constructed by the semihydraulic fill method have had slides during construction.

A fundamental difference in stability during construction thus seems to be indicated. The hydraulic fill method deposits the material from flumes or pipes near the faces of the dam; the larger particles stay there and the finer ones move toward the center, the finest of all going into the central pool and being deposited there. Thus the toe and faces of a dam produced by this method are more pervious, allowing water to drain out from the interior of the dam. Even if most of the drainage from the cores is by vertical crater action, as some engineers maintain, such action takes place not only in the portion of the core underlying the central pool but also in those portions of the core covered by the pervious outer sections of the dam. Hence, whether the main drainage of the core is upward, downward, or sidewise, the importance of pervious outer sections is just as great.

In dams built by the semihydraulic fill method, the toes and faces usually have consisted of car-dump fills. Material is washed away from these fills by jets of water from giants. The finer material goes into a central pool and is deposited, forming the core; the coarser particles are dropped near the car-dump fill. In consequence of this action, the car-dump fill at the face is often more dense and impervious than the material immediately adjoining it on the inside of the dam, for this latter material has had the fines washed out of it by the action of the monitors.

In some cases tests have shown that the material in the car-dumped fills is actually more dense and impervious than that immediately adjoining. Through the presence of the central pool and the sluicing operations, this comparatively pervious area is kept full of water, which exerts considerable hydrostatic pressure on the relatively impervious car fill material at the face. Thus the car fills may form an element of weakness and by imprisoning the water may sometimes be the cause of slides even though the central core of fine material is so stable that it exerts no hydrostatic pressure.

Engineering and economic factors have narrowed the possible use of semihydraulic fill dams so that the situations in which they might be used have become rare indeed. Consequently, very little space is devoted to them here.

* The Alexander Dam (*Earth Dam Projects*, p. 7) is a notable exception to this statement [1].

The stability of hydraulic fill dams has been considered in Section 9, Chapter 21

In Fig 2 is shown the cross-section of the Kingsley Dam in Nebraska. This hydraulic fill dam is more or less typical of many others

10 Typical Hydraulic Fill Construction In typical hydraulic fill dams construction water under heavy pressure is delivered to a giant or monitor,' which is a large nozzle mounted on a standard with a ball-and-socket joint so that the direction of the stream may be readily changed without moving the standard, which is generally weighted down with stones or iron while in operation The giant is directed against the bank which it is desired to excavate in such a manner that the water will undercut the material and then break it up The velocity of the water as it leaves the giant may be from 100 to 200 ft per sec As the force of the water cuts down the material the water and loosened material are guided to flumes or pipes through which they flow to the dam and are deposited

The minimum grade of these flumes or pipes is from 3 to 6%, according to the nature of the material The water for the giant is obtained from high-pressure pumps located at the riverside or else from a pipeline with its source at a point having a much higher elevation Sometimes the material is excavated by suction dredges and pumped to the site through pipelines

Flumes or bench pipes are generally maintained at both the upstream and downstream faces discharging water and materials toward the center of the dam A central pool of water is maintained whose width varies constantly

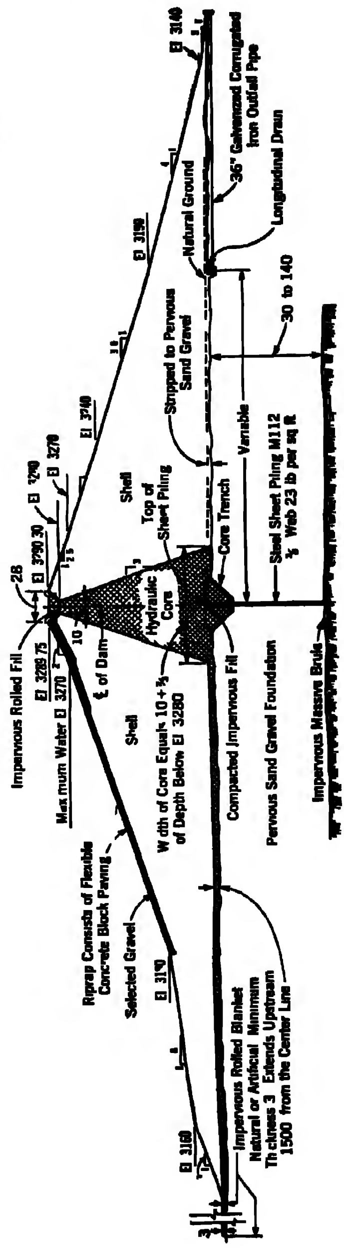

FIG 2 Kingsley hydraulic fill dam Ogallala Nebr

between maximum and minimum widths prescribed directly or indirectly by the requirements of the engineer. As the water discharges from the sluices or trestles, the coarser material is deposited near the faces and the finer material flows into the central pool with the water and is slowly precipitated. In general, there is a gradation from the coarsest material near the faces of the dam to fine sand at the shoulder of the core pool, the extremely fine material being deposited in the core pool. In many cases, however, some of the coarsest material is deposited near the faces, but the body of the shell is a mixture without the nice gradation of sizes which should theoretically be found in the shell from coarse to fine as the core pool is approached.

Although in all hydraulic fill dams the material is carried on to the dam by water and deposited, there are several different ways in which the material may be excavated and transported before it reaches the dam. At the borrow pit it may be excavated by hydraulic giants, by dredges, by steam shovels, or by drag lines. In the latter two cases the material is transported dry to a hog box where it is mixed with water and then run or forced through pipes to the beach pipes on the dam.

11. Materials Suitable for Hydraulic Fill Dams. Not all materials are suitable for use in the construction of a hydraulic fill dam. Fine materials which are very uniform in size should not be utilized for this purpose as they are likely to be deposited in the shell at a density less than that at critical void ratio. In this condition a uniform material would be subject to a flow slide. Also soils composed almost exclusively of very fine particles, such as clays and silts or silts and fine sands, cannot be used in making hydraulic fill dams as the particles will not settle and consolidate promptly enough.*

Accordingly, it is desirable that the materials in the shells of hydraulic fill dams should be nonuniform in character. Glacial deposits frequently provide materials which are ideal for the construction of hydraulic fill dams. The materials range all the way from clay and rock-flour sizes up to stones and boulders too large to be conveniently transported by hydraulic methods. Cobble Mountain and Winsor Dams in Massachusetts, the Miami Dams in Ohio, and Kingsley Dam in Nebraska are all built of glacial sands, gravel, stone, silts, and clays.

An ideal borrow-pit material for a hydraulic fill dam might have 15 to 30% of fine material (silt and fine sand) from 0.005 mm to 0.15 mm, most of which will be for the core and the rest of the material grading upward from fine sand 0.15 mm to cobbles 6 in. or more in diameter. Between these extremes there is a considerable range of materials which may be successfully utilized for hydraulic fill dams.

Colloidal material in a core should be avoided; even if it is present in the borrow pit, it can be wasted from the core pool. The coarse clays and silts will make satisfactory cores.

* For the story of an unsuccessful attempt to build a hydraulic fill dam almost exclusively of very fine material, see p. 7 (Alexander Dam) of Ref. 1, Section 20.

It was found by the late Allen Hazen [7] and others that when the effective size of the core material is not less than 0.01 mm the core consolidates in a very satisfactory manner but that if it contains a high percentage of very fine colloidal material it does not consolidate perceptibly during construction. Thus it is sometimes desirable to waste the finest of the fines, but if all the fines are extremely fine, it will be necessary to use them and to consider the resulting increased core pressure in the design.

12. Desirability of Narrow Cores. Accidents have occurred in the past owing to the fact that shells were made too narrow to resist the semiliquid pressure of the core. In general it is desirable to have the hydraulic fill core as narrow as practicable down to a core slope of 3 on 1 or 4 on 1. Shell material, i.e., material outside the core, has a much greater shear strength than the core material. Therefore, other things being equal, a higher factor of safety against the internal pressure of the core will result by increasing the amount of shell material, decreasing the amount of core material, and steepening the slope of the core (see also Section 10, Chapter 21).

An important point about the relationship of the core and shell of a hydraulic fill dam is that the shell should be tremendously more pervious than the core. If the shell is 100 times as pervious as the core, the necessary requirements of stability and relative permeability will have been met. Many shells are at least 1000 times as pervious as the cores. The seepage through the core is practically never of appreciable economic importance even in hydraulic fill dams used for water supply.

In view of the above, if only 10 or 15 ft of the central portion of a hydraulic fill dam is true core, this is generally enough to insure adequate watertightness. The greater width of the core is just to insure obtaining a portion of the core at the center which is free from sand intrusions.

The minimum width of the core should be not less than 20 ft because it is usually impracticable to construct a core narrower than this and be sure to avoid getting sand lenses through it. For this reason it is usual to require that the top 25 or 30 ft of the hydraulic fill dam or at least its core be constructed by rolled fill methods.

13. Settlement of Earth Dams. Settlement depends on the character of the material in the embankment and the foundation and on the methods of construction. It is customary to construct earth dams to a somewhat greater height and width than the neat dimensions called for by the plans. For an embankment rolled in 6-in. layers of impervious and pervious materials, there is no reason for anticipating any appreciable settlement in the embankment itself, but settlement may occur in the foundation. For a rolled fill dam on an unyielding foundation a nominal allowance for settlement of 1% is sufficient.

For hydraulic fill dams, settlement due to consolidation of the core must be anticipated and will vary with the character of the material. With modern methods (narrow cores and wastage of the excessively fine fines,

including colloids), the allowance for settlement of the embankment (after completion) need never exceed 4%. This is exclusive of foundation settlement.

Foundation settlements have ranged all the way from practically 0 up to 8% of the height of the dam or more, most of which takes place during construction. Deep plastic foundations of relatively unconsolidated clays show the greatest settlement.

Considering settlement of both foundation and embankment, the total provision which should be made for settlement after the completion of the dam will range from a nominal 1% of the height of the dam to a maximum of 6% or more. Thus if 6% had been determined as the proper allowance for settlement after completion and the dam was 100 ft high, it would actually be constructed to an elevation of 106 ft above the original foundation elevation.

14. Approximate Quantities in Earth Dams. In connection with preliminary investigations it is frequently necessary to make a number of preliminary estimates of cost of earth dams. Accordingly, in Table 1, there are given a number of formulas for finding the volume of earth dams for various heights and slopes.

TABLE 1

FORMULAS FOR VOLUME OF EARTH DAMS, IN CUBIC YARDS PER LINEAR FOOT *

[Top width equals 0.25 of the height of the dam]

Slope of One Face	Slope of Other Face	Volume
1 on 2	1 on 2	$0.0833h^2$
1 on 2.5	1 on 2	$0.0926h^2$
1 on 2.5	1 on 2.5	$0.1021h^2$
1 on 3	1 on 2	$0.1020h^2$
1 on 3	1 on 3	$0.1203h^2$
1 on 3.5	1 on 2	$0.1110h^2$
1 on 3.5	1 on 3	$0.1296h^2$
1 on 4	1 on 2	$0.1203h^2$
1 on 4	1 on 2.5	$0.1295h^2$
1 on 4	1 on 3	$0.1387h^2$
1 on 4	1 on 4	$0.1574h^2$
1 on 5	1 on 5	$0.1944h^2$
1 on 6	1 on 6	$0.2313h^2$

* h equals height of dam.

15. Slope Protection for Earth Dams. The upstream slope of an earth dam always requires protective cover against erosion due to wave action. The top and the downstream slope of an earth dam which is exposed to the elements must be protected against erosion from wind and rain. Usually a protective cover which is designed to be safe against wave action is ample for protection against the effects of wind and rain.

The usual types of protective covers are:

1. Dumped stone riprap.
2. Hand-placed riprap.
3. Grouted riprap.
4. Concrete slabs and blocks.
5. Bituminous paving.
6. Planting on slopes.

Good stone riprap on a filter bed, discussed in Section 16, is considered superior for the protection of the upstream face of an earth dam from wave action. Slabs, blocks, and monoliths of concrete, discussed in Section 17, are also used for the upstream face. Bituminous paving built in place by the penetration method has been used to withstand mild wave action.

On the downstream face of earth dams, planting with seeding grass or other crops, sodding, sprigging with Bermuda grass, or planting with vines is generally done for protection against relatively mild erosive action, particularly above maximum tailwater elevation. However, if rock or coarse clean gravel is plentiful and close at hand, such protection is often more economical than using a grass cover and topsoil, which are a continuing expense to maintain.

In reservoirs that are emptied frequently, the upstream slope protection should begin at the upstream toe and extend to the top of the dam. However, if the reservoir will not be drawn down below a certain elevation, the slope protection need not go more than a few feet below this elevation. At this elevation the protective cover should be keyed into a berm (Section 18) or be provided with a toe wall of large stones or concrete.

16. Riprap. Riprap must be laid on a well-graded filter bed (Section 20, Chapter 20) and may be one of three types: dumped, hand-placed, or grouted riprap.

Dumped riprap consists of stones dumped in place from cars or trucks, or tossed in place by hand. It is the most desirable type of slope protection because of its self-healing characteristics and its relative economy. This type of riprap can readily adjust itself to any settlement in the dam.

The necessary depth of dumped riprap cannot at present be based on any mathematical analysis or on the results of extended studies of existing riprap, which are badly needed. Opinions differ on this subject. It is suggested that, for estimating purposes, the depth of dumped riprap be assumed to be equal to 60% of the height of the waves calculated by the Stevenson-Molitor formula (Section 10, Chapter 17).

Usually, dumped riprap varies in thickness from 1.5 to 3 ft or more, the thicker and heavier riprap being used for the longer fetches and greater wind velocities. The stone used for dumped riprap should grade from pieces having a least dimension of 50% of the thickness of the layer to those only 3 or 4 in. in the smallest dimension.

Figure 3 indicates a typical construction of dumped riprap.

Attention is again drawn to the fact that all dumped riprap should be placed on a suitable filter layer and that the size of the stones used is dependent on the thickness of the riprap

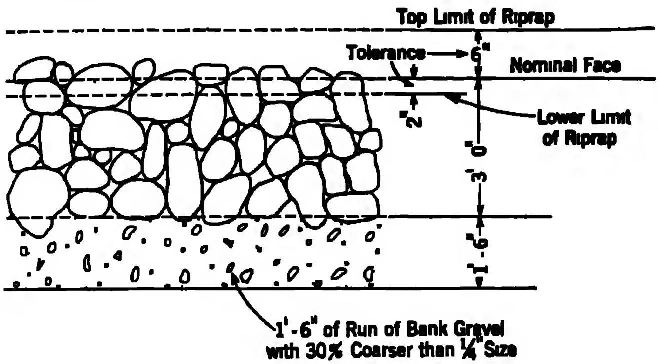

Fig 3 Typical dumped riprap construction

Hand placed riprap consists of stones carefully laid on edge and with the minimum amount of voids The cost per cubic yard of hand-placed riprap is

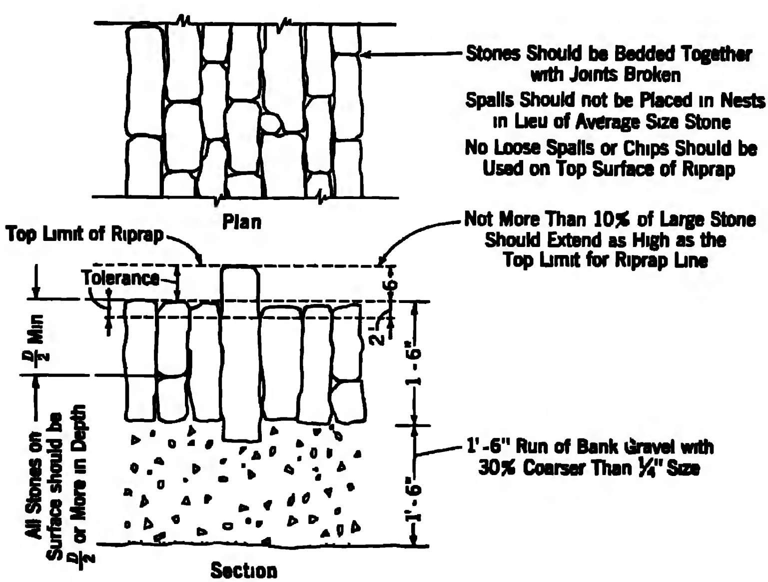

Fig 4 Typical hand-placed riprap construction

considerably greater than the cost of dumped riprap because of the added labor In countries where material is expensive and labor is cheap this type of riprap may be cheaper than dumped riprap Approximately 18 in of

hand-placed riprap laid like dry rubble is considered equivalent to 3 ft of dumped riprap. A minimum thickness of 18 in. is generally suitable, and a common requirement is that at least 50% of the surface shall be stones which are 18 in. deep. Figure 4 indicates typical construction for hand-placed riprap.

Grouted riprap is hand-placed riprap broomed or rodded with a cement mortar. Such riprap is not as stiff as a concrete slab (Section 17) nor is it as flexible as the other types of stone riprap. In the authors' opinion there are few occasions which justify the use of grouted riprap.

Not all stone is suitable for use as riprap. Stone for riprap should be hard and durable and should not break down on long exposure to water, frost, and air. Most of the igneous and metamorphic rocks and many of the sandstones and limestones make excellent riprap. Shales are generally entirely unsuitable. In investigating available rock for riprap, records of service of the rock in the locality should serve as a guide to determine its suitability.

17. Concrete Lining of Upstream Slopes. Concrete linings for upstream slope protection deserve consideration where suitable riprap is not readily available. Such linings may be one of two types.

(*a*) *Monolithic Concrete Lining.* This type consists of a continuous reinforced-concrete slab over the entire surface, with the steel reinforcement in each direction equal to 0.5% of the cross-section area

Many dams in the West have a monolithic concrete slab on the upstream face as the sole protection against wave action. This slab sometimes also furnishes the impervious member of the embankment. Usually a 6- or 8-in. slab is considered sufficient. The practice has a good chance of success in arid climates where the embankment supporting the slab is free-draining sand and gravel.

(*b*) *Concrete Lining of Square Blocks.* In some cases the upstream face of earth dams has been protected from wave action by concrete linings of square blocks, generally not larger than 6 by 6 ft. It is not usually necessary to reinforce these blocks. The thickness of the block in inches should be the same as the dimension of the block in feet; i.e., a block 6 ft square should be 6 in. thick. The squares should be poured alternately, the blocks being separated by layers of three-ply tar paper so that they will adjust themselves to the surface of the embankment in case of settlement. Sometimes precast concrete slabs of much smaller size are used.

If the concrete paving is monolithic or if the concrete blocks are set without spaces between them, it is essential to provide numerous gravel-backed weep holes through the concrete (equal to at least 15% of total area), unless the entire embankment is pervious, in order to allow the water in the embankment to drain away when the reservoir is drawn down quickly. Otherwise, hydrostatic pressure behind the concrete lining might break it or cause it to slide down the slope and possibly, also, cause the sloughing of some of the saturated embankment material.

In the design of either type of concrete slope protection it is generally important to consider also (1) stresses set up by unequal settlement of underlying material and (2) the severity of the climate and durability of the concrete.

18. Berms. In earth dams more than 30 ft high, berms from 6 to 20 ft wide are often used on the downstream face. In high dams berms should be used for about each 30-ft difference in elevation. The main function of berms is to minimize the erosion from rainstorms, the effects of which may be very severe. The outer edge of berms should be higher than the inner edge, in order to prevent rain water from flowing over the edge and down the slope.

A gutter should be placed at the inner edge of the berm and given a slight grade to conduct the storm water to the side of the valley, where other gutters or storm drains conduct it to the toe of the dam. In many of the largest and highest earth dams, the storm water from the berms is collected by catch basins and conducted through storm sewers to the main drainage system at the downstream toe of the dam. If rock or clean heavy gravel is used for protection on the downstream face, it is unnecessary to use berms for drainage.

On some dams, berms are built on the upstream face, and a berm should always be used as a shoulder against which to build the bottom of the riprap.

19. Pipes or Conduits Passing through Earth Dams. The best practice for passing pipes or conduits through earth dams is to place the pipes

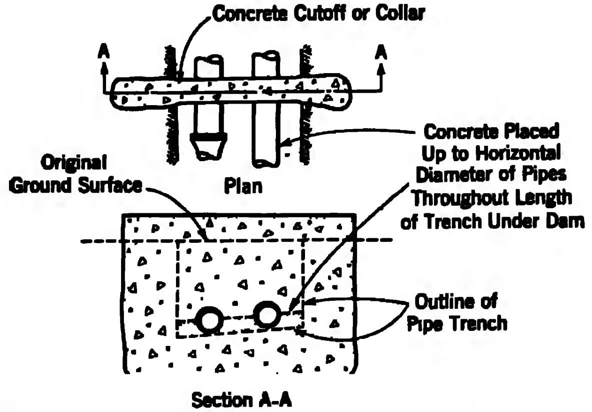

FIG. 5. A suitable method of protecting pipes passing under dam.

and conduits in trenches excavated in the original foundation material or in a trench or tunnel through rock excavated through the hill at the side of the dam. Where pipes are used they should be embedded in concrete placed in the pipe trench at least up to the horizontal diameter of the pipe, in addition to being provided with suitable collars in the manner indicated in Fig. 5.

Control of outlet pipes or conduits should always be from the reservoir side by means of suitable valves or gates. With this arrangement it is possible to repair defects that may develop in a conduit.

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CHAPTER 23

ROCK-FILL DAMS *

1. General. Rock-fill dams are used rather extensively in remote locations where the cost of cement for a concrete dam would be high, where suitable materials for earth dams are not available, and where suitable rock can be quarried at or near the dam site. Attention is called to the fact that a foundation might be acceptable for a rock-fill dam but not for a concrete dam. A number of rock-fill dams have been built to more than 200 ft in height. For example, the Salt Springs Dam [2] shown in Fig. 2 is 328 ft high, and other, much higher projects are contemplated. Rock-fill dams must, of necessity, include an impervious element, which may be either an internal core, as in Figs. 5 and 6, or an upstream impervious facing of concrete properly supported, as in Fig. 2.

The usual type of rock-fill dam has three fundamental parts: (1) the dumped rock fill, (2) an upstream rubble cushion of laid-up stone bonding into the dumped rock, and (3) an upstream impervious facing resting on the rubble cushion. In some dams the rubble cushion is replaced by a graded filter, and the impervious diaphragm on the upstream face is replaced by a relatively heavy earth fill. Such dams, however, are usually referred to as "composite dams" and are discussed in Section 10 of this chapter.

Rock-fill dams are, in general, empirically designed. However, there are certain rules of design which must be applied. The stability of rock-fill dams is analyzed in the same manner as that of earth dams (Chapter 21).

2. Foundation. It is desirable and usual practice to clean off the foundation so that ledge rock is exposed (Section 3*a*, Chapter 25). If this is not done and the rock is deposited directly on the overburden then it will be the shear resistance of the overburden which governs the stability of the rock-fill dam, instead of the shear resistance of the rock fill, which would govern stability if the rock fill were placed directly on ledge rock.

The essential condition for the foundation of a rock-fill dam is that it shall not be subject to material settlement or to erosion from such seepage as may pass through or under it. To prevent such seepage, a cutoff wall at the bottom of the facing may be used. It should be bonded into firm ledge rock or other suitable impervious material. If the foundation is anything

* Abridged from the data compiled by the late Carl Ashley in Chapter 20, *Engineering for Dams*, by William P. Creager, Joel D. Justin, and Julian Hinds, John Wiley & Sons, New York, 1945, Vol. 3.

other than sound ledge rock, the possibility of a foundation blowout must be carefully investigated and guarded against. The cutoff wall should extend across and up the sides of the canyon. Grouting may be required in the ledge rock below the cutoff in order to seal the dam against seepage under the structure.

The connection between the impervious diaphragm on the upstream face of the dam and the cutoff wall should be flexible so that a material amount of movement in the slab may take place without causing a rupture which would produce extensive leakage. The problem is particularly difficult at the junction of cutoff walls up the sides of the canyon with the flexible concrete facing, and the design at this point should permit a large amount of movement without danger of fracturing the concrete facing. The cutoff wall should be able to take any thrust which may be transmitted to it from the impervious diaphragm on the upstream face. There is usually little or no settlement at the cutoff, but a short distance away from it the settlement may be considerable.

3. Face Slopes. Modern rock-fill dams are usually built on a ratio of base to height ranging between 2.5 and 3.0. The downstream slope is usually made

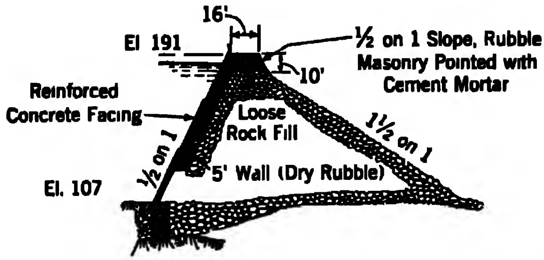

FIG. 1. Section through Beaver Park Dam, Colorado. (*Eng. News*, Vol. 73, p. 660.)

the natural slope of rock dumped from cars or trucks, or about 1 on 1.3 to 1 on 1.4. If the slope is steeper or flatter than this it usually requires additional handling. The upstream slope ranges from the natural dumped rock slope to about 1 on ¾. Slopes as steep as 1 on ½ have been used, as in the case of the Beaver Park Dam in Colorado (Fig. 1), but later practice is to build the upstream slope with the natural dumped rock slope of about 1 on 1.3, as in Fig. 2. Both conservatism and economy usually favor the upstream face with natural slope.

The upstream face is usually made concave along the water slope to prevent buckling of the facing when settling occurs. Dams exceeding 100 ft in height should have a crest width of not less than 15 ft, and even low dams should not have a crest width of less than 10 ft. The water pressure on a rock-fill dam is resisted only by the weight of the rock. No arching or cantilever action can be considered as aiding stability.

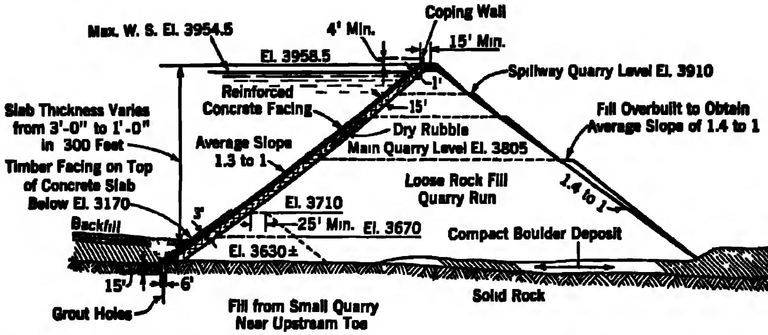

FIG. 2. Salt Springs Dam, Mokelumne River, California. Maximum cross-section. (Pacific Gas and Electric Co., San Francisco.)

4. Safety against Sliding. Almost any rock-fill dam with an impervious upstream face which it would be practicable to build on a suitable foundation will necessarily have a relatively high factor of safety against sliding because of the large mass involved. Thus J. D. Galloway [1] points out that, "On the assumption of a dam 200 ft high with a crest 15 ft wide, a downstream slope of 1 on 1.4 and a unit weight of 100 lb per cu ft for the loose rock and with the water load, ratios of height to base of 1:2.25, 1:2.5, and 1:3 would have sliding factors (ratios of weight of rock to water pressure) of 4.50, 5.14, and 6.45, respectively. These ratios are practically constant for all heights of dams." Nevertheless the adequacy of the foundation should always be investigated.

Tests have been made to determine the sliding friction angle for various types of rock on ledge rock. It is seldom less than 45 degrees.

5. Main Rock Fill. The rock fill in the main part of the dam must be of sound rock which will not readily disintegrate, split, or crush. Thus, shales which slake in the presence of air are dangerous and should be rigidly excluded. Rock which, when shot, shatters into very small pieces with a high percentage of chips and dust is unsuitable.

Methods of placing rock fill vary with height of canyon walls and location of quarries. Derricks, cranes, or cableways are used chiefly when the rock fill is obtainable directly at the abutment ends of the dam Frequently, in large rock-fill dam construction, railroad cars and trucks transport the rock fill.

Considerable difference of opinion exists as to the height of drop, or lift, which is advisable. At Salt Springs (Fig. 2), with granite, a maximum drop of 165 ft was used, and the settlement in the dam proper was relatively small. The practical limit would be such that it would not seriously injure the rock or be dangerous to workmen.

The size of the individual rocks may vary greatly. With ordinary dump trucks the maximum size is about 3 tons. Using air-dumped railroad cars or

very heavy trucks it may average 10 tons, reaching a maximum size in recent dams of about 25 tons. The sizes if placed by derricks, cranes, or cableways depend upon the economical machine capacity. As far as practicable, chips and dust should be excluded from the fill. Certainly, 10% should be the top limit for such objectionable material under most conditions.

6. Rubble Backing of Impervious Face. If a concrete slab face is used, rubble should be placed as a cushion on the upstream face of the dam and for chinking up large voids, thus forming a substantial backing for transmitting and distributing the water load on the impervious face to the main body of the dam. Also the use of a rubble backing has, in some dams, permitted an extremely steep upstream face (see Fig. 1).

7. Impervious Upstream Facing. As previously stated, most recent practice in making the rock-fill dam watertight tends to the use of an impervious facing on the upstream slope attached in such a manner as to obtain some degree of flexibility to the cutoff wall at the upstream toe. The facing can be made of wood, steel, or concrete. In a few dams it has been of bituminous concrete.

Timber Upstream Facing. The older dams were usually faced with wood, and its flexibility, which permits the rock dam to settle without significant damage, is quite an advantage. It is used frequently in remote locations where timber is plentiful.

Below the minimum water surface elevation, timber facings are practically permanent. Above this elevation, the life of a timber facing may be relatively short. Some timber facings have been built of timber impregnated with creosote, as on the Sabrina and South Lake Dams on Bishop Creek, California [1], which were resurfaced with pressure-treated Douglas fir and redwood. Such facings have a relatively long life, even in storage reservoirs where a large portion of the facing is exposed to the air for a material part of the time.

Timber facings are subject to a serious fire hazard at any time when exposed, but this hazard may be largely obviated by the addition of a layer of Gunite on wire mesh attached to the timber face or by an efficient patrol system. Timber facings are often economical in remote locations and are particularly applicable where the variation in the elevation of the water surface is not great. These facings are usually from one to three layers of 2 by 12 in. or 3 by 12 in. plank laid parallel to the dam axis and spiked to sills, say 8 by 8 in., imbedded in and anchored to the rubble cushion.

Steel Facing. Steel plates have been successfully used to provide the impervious membrane on the upstream face of rock-fill dams, as in the Penrose-Rosemont Dam, Colorado [3], and the Skaguay Dam of Southern Colorado Power Co. [4]. Expansion joints of the U type are generally used, and some means must be adopted for anchoring the plates to the rubble backing. The plate joints may be riveted and calked or welded. Facing plates range from ¼ to ½ in. in thickness and should be of copper bearing steel to reduce corrosion. Protection from the action of water and air is secured by painting

all surfaces with bituminous paints. After more than 45 years of use, the Skaguay Dam, which was painted on the face only, shows only very slight corrosion.

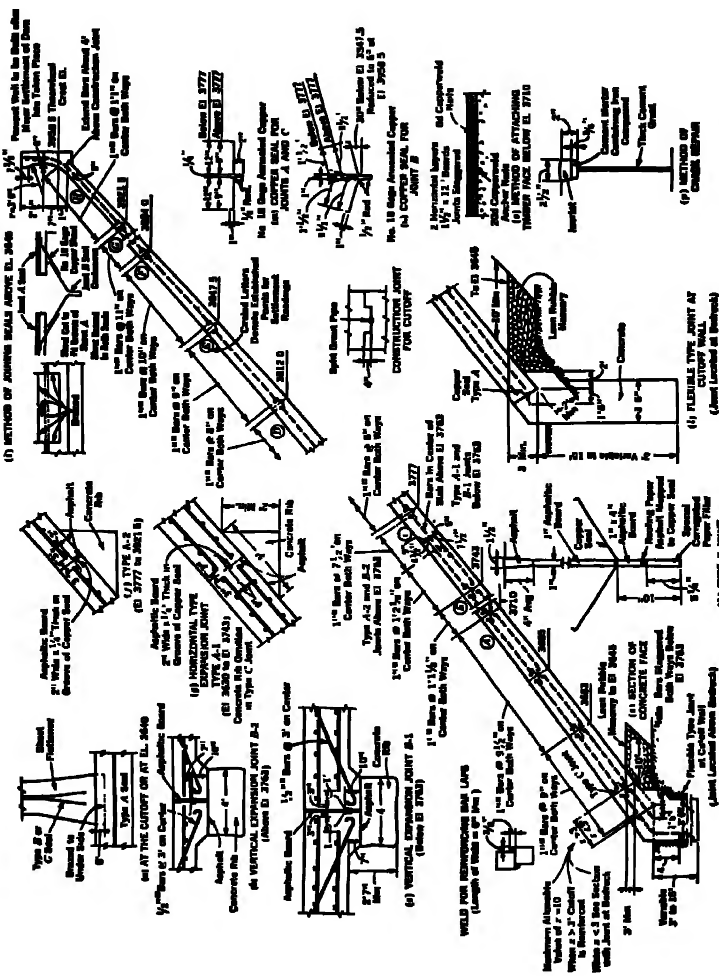

FIG. 3. Details of concrete face and cutoff wall. Salt Springs Dam, California.

Concrete Facing. The general practice is to use reinforced concrete to provide the impervious membrane on the upstream face of rock-fill dams. This membrane may be a single sheet of concrete or it may be made laminated with two sheets. Recent practice of the single sheet has secured good results and may now be considered standard practice in the United States.

It has been found that a concrete facing can be made safe against water pressure up to 300 ft with a thickness equal to about 1% of the height with a minimum thickness of 12 in. [1]. The reinforcement is usually made the same in both directions and equal to about 0.5% of the cross-section of the slab. It is usually placed in the center of the slab except where thickness exceeds about 2 ft, in which case two layers may be used, one near each face.

The concrete may be placed in 30 to 60 ft squares with expansion joints to permit adjustment under settlement of the rock fill, or it may be poured as one large mat with construction joints only between pours and with reinforcing continuous across the joints. The relative length of the upstream facing to its vertical height plays an important part in determining the method of pouring the facing slab. In high dams in narrow canyons, the facing should be divided into squares by expansion joints to accommodate the larger settlement movements to be expected.

At Dix River Dam [5], Salt Springs [1, 2], and other dams it has been the practice to construct grooves in the rubble cushion approximately 2 ft deep by 3 to 4 ft wide along the lines of vertical and horizontal expansion joints. The horizontal and vertical concrete stringers on which the slabs rest were poured in these grooves without the use of bottom forms so that they would bond thoroughly into the rock fill. They were not reinforced. Their top was finished to line and grade to receive the concrete slabs.

Some engineers feel that bonding the slab to the rubble cushion tends to prevent the free action of the expansion joints. However, as described above, it has been demonstrated by actual construction that the concrete facing will adjust itself when poured directly on the rubble cushion. Such facing costs less than any other type, and successful experience justifies its use. Figure 3 shows some of the more important details of the reinforced-concrete facing used at the Salt Springs Dam in California.

8. Settlement and Sluicing. Total vertical settlement in excess of 5% of the height has occurred in some rock-fill dams, and the horizontal displacement may be nearly as great. If, however, the loose rock dump fill is constructed in advance of the rubble cushion with a proper use of sluicing water, the initial settlement may be large; but subsequent settlement, after the placing of the rubble cushion and the impervious facing, should not exceed 2% of the height.

In the Salt Springs Dam, height 328 ft, for instance, provision was made for a vertical settlement of 6 ft and a horizontal displacement of 4.2 ft. The actual vertical settlement at the crest has been about 2 ft. Settlement is not uniform nor is it vertical. The tendency is for movement to take place in a direction perpendicular to the water face. The proximity and configuration of the canyon walls have a marked influence on settlement at any particular point. At Salt Springs Dam the maximum total displacement took place at a distance about 40% up from the base of the total height of the dam.

Small chips and dust, if present to any considerable extent, will lodge between the rocks and will later sift down into the interstices of the larger

rock under the action of rain falling on the dam and passing down through it. This may cause a material settlement of the dam with serious movement and cracking of the impervious upstream face. Accordingly the fines should be constantly washed into the rock mass with hose streams as the fill is being made. The quantity of water which should be used for this sluicing operation will vary with local conditions. Theoretically, if there are no spalls or dust, no sluicing would be required, but practically it has been found desirable to use from 2 to 4 times the volume of the dam in sluicing water. Continuous sluicing during construction will produce presettlement of the fill and thus increase stability. The elimination of lenses of small particles thus obtained will reduce local settlement when water pressure is applied on the dam. Generally speaking, the use of an adequate amount of sluicing water during construction is one of the most important factors in the proper construction of rock-fill dams. In this connection, it is of interest to note that in the original construction of San Gabriel No. 2 very little sluicing was done and as a consequence the first heavy rain resulted in a 12-ft settlement causing the destruction of the concrete facing [6]. At Salt Springs Dam and San Gabriel No. 1 the sluicing water was twice the volume of the rock fill. At Nantahala, N. C., the ratio of water to rock fill exceeded 4.

In addition to the settlement caused by chips and dust sifting down into the interstices of the larger rocks, referred to above, settlement is also caused by the crushing of the bearing points of the rocks when the load comes on them from the weight of the rocks above and from the transmitted water pressure. In this connection it is worth noting that quarry-run rock, which contains assorted sizes, provides more bearing points than big rock alone of uniform size.

The settlement is, of course, much greater in the early life of the dam and is greatest during the first few months. The dam is usually built from the canyon sides toward the center, and as the fill advances from each side it causes the rock to settle toward the center. The initial settlement hastened by sluicing should be allowed to take place to as great an extent as practicable before the construction of the impervious upstream face is started.

9. Spillways and Freeboard. It is very nearly as essential to prevent the overtopping of a rock-fill dam in time of flood as it is an earth dam. Any type of spillway suitable for an earth dam is also suitable for a rock-fill dam. In many cases the "side channel" type of spillway (Section 3, Chapter 26), where a spillway channel is cut through the ledge rock at one side of the dam, has been utilized. Sometimes the discharge from such a spillway is through a tunnel. Other things being equal a "saddle" spillway (Section 4, Chapter 26) remote from the dam itself is desirable.

10. Composite Type of Rock-fill Dam. This type consists of a rock fill on the downstream side of the dam and an earth fill on the upstream side. Such a dam, when properly constructed, produces a very stable and satisfactory structure. The earth fill furnishes the watertight portion of the dam,

an intermediate section provides a filter, and the rock fill forming the downstream portion provides ready drainage for seepage water and adds a greater degree of stability to the structure than would generally be provided by an equivalent amount of earth. Glenville Dam, shown in Fig. 4, is an example of the composite type rock-fill dam.

11. Earth-core Type of Rock-fill Dams. A conventional type of rock-fill dam having an impervious element consisting of an earth core in the center of the dam is indicated in Fig 5. A somewhat unique type in which the core is considerably inclined upstream, as indicated in Fig. 6, was constructed on the Nantahala River in western North Carolina in 1941, by the Nantahala Power and Light Company The Nantahala Dam was designed and constructed under the direction of J. P Growdon, Chief Hydraulic Engineer of the Aluminum Company of America The arrangement of the inclined impervious diaphragm of earth, with filters both above and below it, is believed to be original with Mr Growdon

The earth-core type of rock-fill dam is particularly adapted to locations where there is a plentiful supply of good rock fill and, at the same time, sufficient earth for the core In addition, the earth core possesses certain advantages over the concrete slab or other conventional facings frequently used for rock-fill dams. It can yield readily to settlement of the rock fill without damage; it is self-healing with respect to any cracks which may be formed. Because of this feature dams of this type often have less seepage than rock-fill dams with a concrete face. The seepage at Nantahala, for instance, is only about 6 cu ft per min.

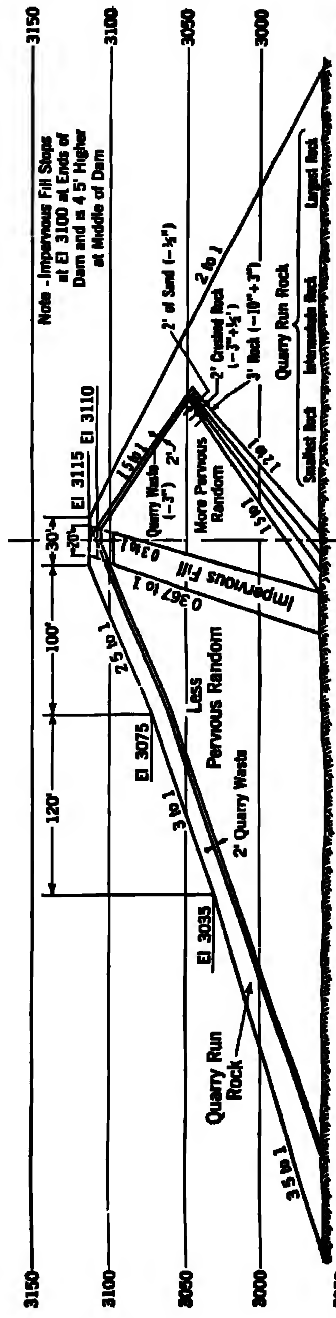

FIG 4. Glenville Dam (Nantahala Power and Light Co., North Carolina)

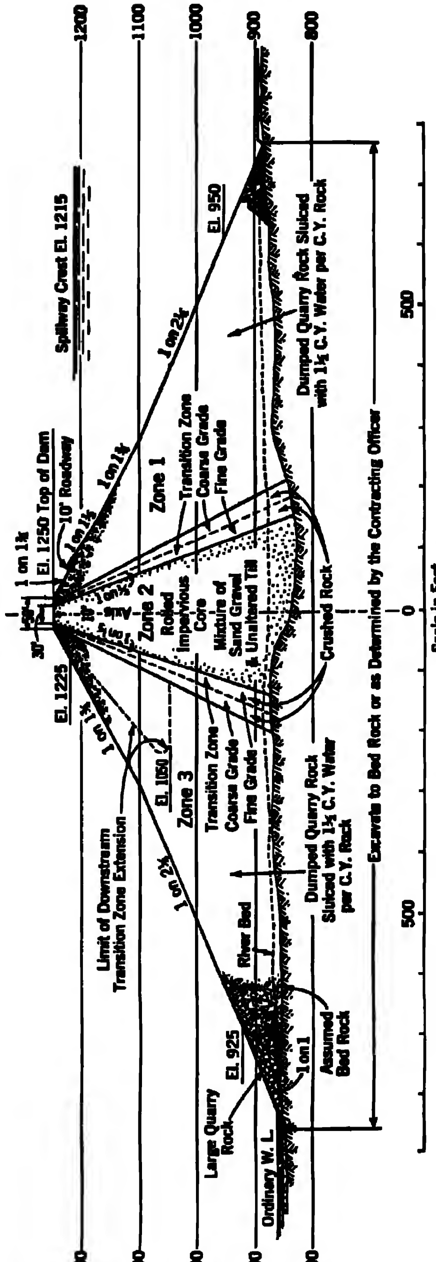

FIG. 5. Stevens Dam. (U. S. Engineer Office, Seattle, Wash.)

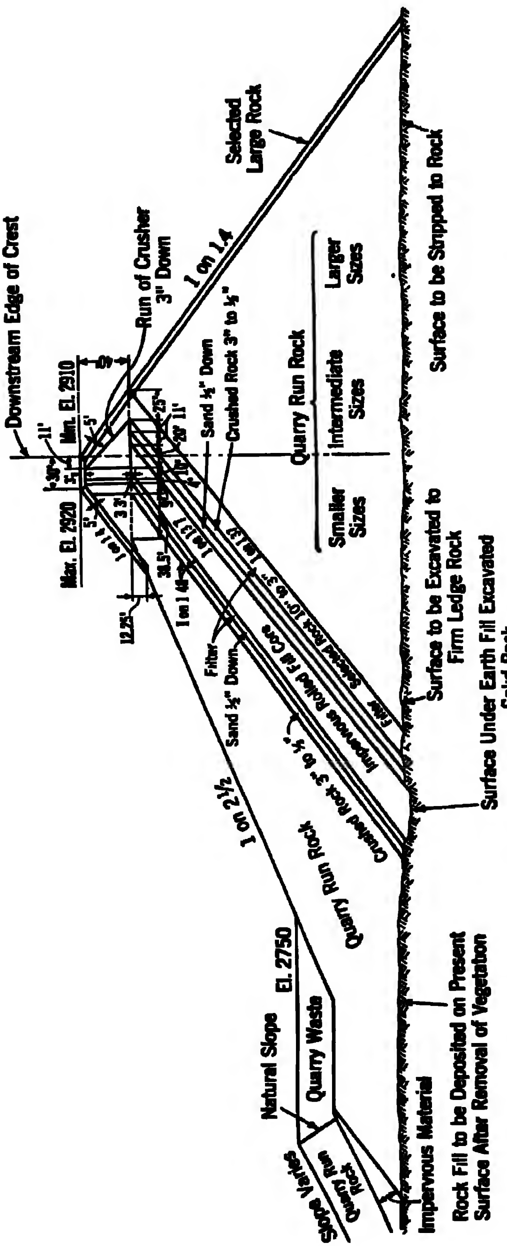

FIG. 6. Nantahala Dam. (Nantahala Power and Light Co., North Carolina.)

As shown in Fig. 6, the core is protected on both sides by filter layers. The downstream filter prevents loss of core material by piping due to pressure of reservoir water, and the upstream filter affords similar protection against reverse flow when the reservoir is drawn down.

The required thickness of core depends on the permeability of the material. When highly impervious fill is available, a very thin blanket would, theoretically, suffice. However, practical considerations also govern, such as adequate space for spreading and rolling and the provision of an ample mass to allow for deformation due to settlement of the rock fill. These latter considerations determined the thickness of the Nantahala core.

Similarly, the thicknesses of the filter layers will, in general, be governed by practical construction requirements. Some "seasoning"—migration of fine particles near the layer boundaries—is certain to take place, and an ample margin must be allowed for this, as well as for the inaccuracies in placing, which are unavoidable in rapid construction. Tests on efficiency of available filter material are usually made (Section 20, Chapter 20).

Considering the general case, it is obvious that the core could be placed anywhere within the section from a position at the center, as in Fig. 5, to a position nearly parallel to the upstream slope, as in Fig. 6. In the vertical position, the water pressure is transmitted horizontally to the downstream rock fill, so that generally, in order to secure adequate stability, the downstream slope must be flatter than the angle of repose. Moving the core from this position to progressively flatter positions upstream permits a steepening of the downstream slope but requires a corresponding flattening of the upstream slope to provide protection against upstream sloughing during drawdown.

In an investigation of the optimum position of the core, Glennon Gilboy has found that shifting the core over the full possible range will not change materially the total yardage of the dam. He concludes, therefore, that economy of design appears to rest on considerations of efficient construction rather than on total yardage.

He points out that locating the core in such a position that the upstream face of the downstream rock fill can be built on its angle of repose will offer the following economies.

(*a*) Sufficient stability against direct water pressure will be obtained with the downstream face at its angle of repose, thus avoiding, for the entire downstream rock fill, any expensive rehandling when placing.

(*b*) The downstream fill can be placed well in advance of the core and take a large part of its settlement before the core is placed upon it.

(*c*) The downstream filter can be placed at its angle of repose, avoiding the use of batterboards or sawtooth construction in this very important element of the dam.

The crucial feature of design of a dam of this type is the stability of the upstream slope when subject to rapid drawdown of the reservoir. Proper

methods for testing for stability under such conditions are described in Chapter 21.

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CHAPTER 24

TIMBER DAMS AND STEEL DAMS

1. General. Timber dams and steel dams are not very extensively used in the United States. Increased costs of both timber and steel have made other types of dams more economical. Occasionally conditions may still favor the construction of timber dams where timber is plentiful and transportation difficult. A change in the relative price of steel may again bring the steel dam into popularity. Because of a foundation failure of the Hauser Lake Steel Dam [4], which had nothing to do with the fact that it was a steel dam, this type of dam has an undeservedly bad reputation. Only three important structural-steel dams [1, 2, 3, and 4] have been built in the United States.

Under some conditions a multiple-arch or slab-type buttressed dam made entirely of steel except for a concrete foundation, with face plates of stainless steel and buttresses made up of structural-steel columns, struts, and braces, might compete with a concrete structure of similar type, provided that engineers were willing to consider it. With proper attention to maintenance, especially to the painting of the exposed structural steel, such a structure should be just as durable as the concrete dam.

A general outline of the forces acting on dams and the requirements for their stability is given in Parts B and C of Chapter 17.

A. Timber Dams

2. Advantages and Disadvantages of Timber Dams. The life of a well-built timber dam has been variously estimated at 20 to 30 years. However, it is usually difficult to estimate the life of a structure that is properly maintained. Dams reputed to be 80 to 100 years of age have been cited, but probably a very small percentage of the original timber remained in such dams.

The maintenance charges for timber dams are large, particularly at sites where large floods and ice runs are frequent. Leakage is frequently very great, and the leaks are often exceedingly difficult to repair if the dam is relatively high and if a drawdown of the pond for repairs seriously affects operation. The large maintenance charges and leakage have created a prejudice against this type of dam. There are doubtless many instances, however, in which a concrete dam has been constructed when true economy would have dictated the selection of a timber structure.

3. The A-frame Type. Figure 1 shows a type of timber dam known as the A-frame type. It is generally built of squared timbers and planks and is not rock-filled. For its stability it depends on the weight of water on its deck and the anchorage of the sills to the foundation. It is probably the ancestor of the reinforced, flat-deck, hollow type of concrete dam. The deck makes an angle of 30 degrees or less with the horizontal.

The sills, *a*, are first fastened to the ledge rock by wedge bolts or anchor bolts, preferably grouted in. The struts, *b*, are then framed to the sills and held in place by cross-bracing and batten blocks. The wales, *c*, are then placed, the entire structure being thoroughly drift-pinned together. These

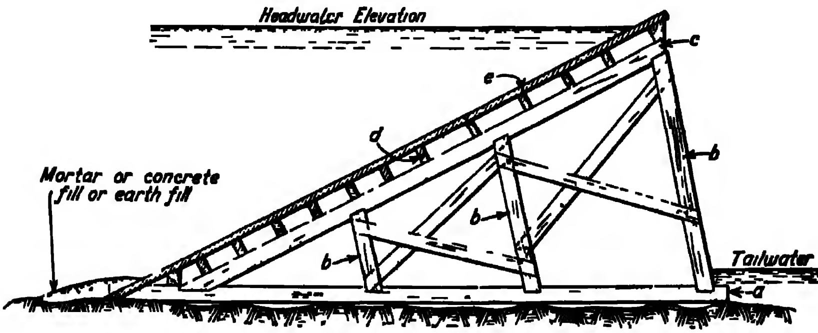


FIG. 1 A-frame type of timber dam

bents are placed from 6 to 12 ft apart, according to the height of the dam and the size of timbers used. Across the bents are placed the studs, *d*, to which the lagging, *e*, is nailed. The lagging should be either tongued and grooved or lapped and should not be less than 2-in. stuff.

4. The Rock-filled Crib Type. In this type of timber dam, cribs of round or squared timbers are drift-bolted together, filled with rock fragments or boulders, and topped by a plank deck. The timbers are usually spaced about 8-ft centers both ways. The bottom timbers of the cribs are often pinned to the rock foundation if the site is not submerged. Figure 2 shows a typical dam of this kind; but many different forms have been adopted.

For rock foundations, the shape of section indicated in Fig. 2 is frequently altered to resemble that of the A-frame type, in order to take advantage of the weight of the water on the sloping deck. This procedure obviates the necessity of the rock fill for stability against overturning, but the close cribwork provides for a more substantial body of the dam than that indicated in Fig. 1. As in the A-frame type, it will then be necessary to anchor the base to the rock to prevent sliding. For low dams on soft foundations, where erosion from overflow would be serious, this section is usually reversed, having a nearly vertical upstream face and a long, sloping downstream face, frequently stepped, in order to drop the water without great disturbance. Between these

extremes many shapes of section have been adopted, some having both upstream and downstream faces sloping or stepped as in Fig. 2.

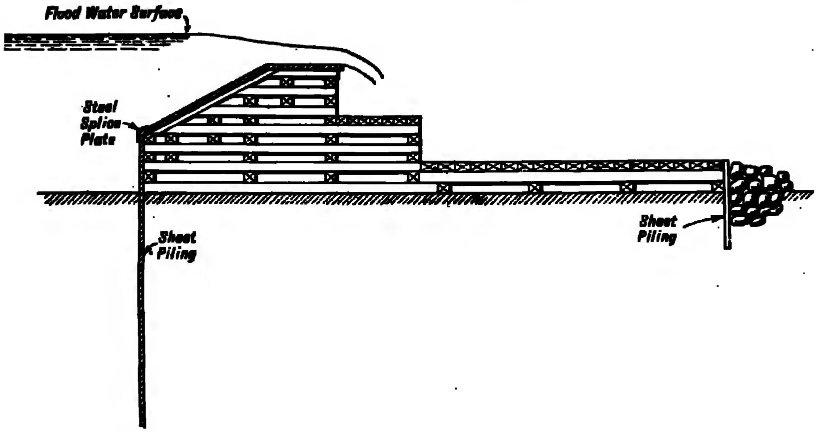


FIG. 2. Rock-filled crib type of timber dam.

5. The Beaver Type. Another type of timber dam, which is used infrequently and only for low heads, is the beaver-type dam. Round timbers are used for the bents as in Fig. 3. The upstream slope of such a dam should not

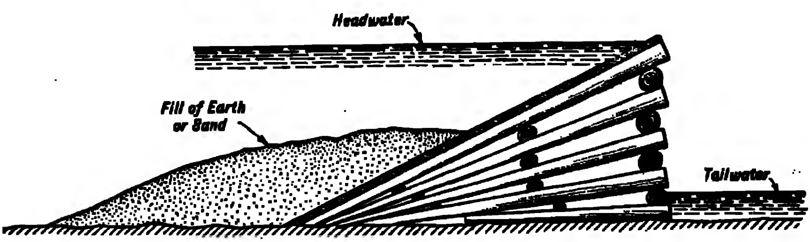


FIG. 3. Beaver type of timber dam.

be steeper than 1 on 2. The butts of the timbers all point downstream. Between the butts are placed spacer logs, which are drift-pinned to the other logs. Also, the tips of the timbers pointing upstream are drift-pinned together and the bottom timbers are fastened to the foundation with anchor bolts, if possible. There is usually a plank deck. Sometimes a mat of brush or the branches of the trees take the place of the plank deck.

6. Stability of Timber Dams. The theory of design of masonry dams, given in Chapter 17, is also applicable to all types of timber dams. While uplift is not present in any type of timber dam, the submerged weight should be used below the elevation of tailwater. The effective weight of submerged rock fill is

$$w_s = (w - 62.5)(1 - p) \qquad [1]$$

where w is the weight of 1 cu ft of solid stone in air and p is the percentage of voids in the fill. It must be remembered that the percentage of voids in the stone fill, in the small spaces between the layers of timber in the crib-work, may be very large, depending upon the care with which the fill is placed and the relative size of the timbers and the large stones. The buoyancy of the timber depends, of course, on the kind of timber used. Frequently as much as 25% of the entire volume of a rock-filled crib dam is composed of timber.

On account of the relative lightness of timber dams, the dimensions necessary to prevent sliding are almost always sufficient to prevent overturning. For stability against sliding, the effective weight of a rock-filled crib dam on a rock foundation, including the vertical water pressure on the upstream face, ranges from 2.5 times the horizontal pressure of the water for unimportant, temporary structures on rough foundations to 4 times the horizontal pressure for important dams on smooth foundations.

The stability of A-frame and beaver dams, which have no rock fill to prevent sliding, depends almost entirely upon the strength of the pins that fasten the dams to rock foundations, unless the foundation is so rough as to permit a horizontal support for the bottom timbers. Friction of wet timber on stone is very small.

The factors of safety to prevent sliding of timber dams on earth foundations follow closely those recommended for masonry dams on earth.

The timbers of the dam should be investigated for strength to transmit the loads to the foundations. In rock-filled dams, much of the load is transmitted through the rock fill, thus relieving the stress on the lower timbers.

7. Tightening the Foundation. If the dam rests on a rock foundation, the lagging at the upstream toe should be framed as closely as possible to the rock and the junction should be properly sealed. In some dams the rock is carefully cleaned and a layer of concrete deposited against the toe, as indicated in Fig. 1. In others a fill of impervious earth is deposited against the upstream face of the dam, provided the velocity during floods is not sufficient to disturb it. Low dams on silt-laden streams may have a tight layer of sediment deposited against them during the first freshet.

Without adequate sheet piling at the upstream toe, timber dams on earth foundations are precarious even though an impervious fill is placed above the dam. Great care should be exercised to obtain a tight bond between the top of the piling and the lagging of the deck, and a splice plate of steel thoroughly fastened by lag screws is advisable, as a slight movement of the dam is likely to loosen the junction. It is also advisable, where sheet piling is used, to provide a vertical upstream face at least 4 or 5 ft high and allow the sheet piling to lap this face completely, in order to afford better opportunity for fastening it to the dam. This arrangement is shown in Fig. 2.

For further general information on foundation treatment refer to Chapter 25.

8. Protection against Erosion. Spillway dams must be protected against erosion from the overflow, if the foundations are soft. This is usually accomplished by sloping or stepping the downstream face, as indicated in Fig. 2, and providing an apron to protect the foundations. The apron should be a low rock-filled crib with sufficient rock above the bottom timbers to prevent flotation; or the apron may be anchored to round piling. As shown in Fig. 2, a row of short piling and a fill of large rock fragments protect the lower end of the apron from being undermined. (See also Section 5, Chapter 25.)

9. Choice of Type. The beaver type of timber dam is the lowest in cost if plenty of timber is available. With more expensive timber, the A-frame type is usually the cheapest. Brush-topped beaver dams are seldom used for permanent structures.

The advantage of the beaver and A-frame types over the rock-filled is found in their smaller first cost and lower maintenance charges. Rock-filled dams are hard to repair, as the timbers, being buried in the fill, are difficult to replace. The greatest objection to the A-frame type is its danger of failure when neglected. The rock-filled dam is in a large measure supported by the fill and will stand some time after the timbers have become materially decayed.

In the usually remote contingency of nearly complete submergence, which results in negligible head on the crest, the beaver and A-frame dams are likely to float. Neither of these types is easily constructed in deep water, while crib dams can be partly constructed on land, floated into place, and sunk by filling with stone.

The A-frame type is not particularly suited to earth foundations requiring sheet piling, as the desirability of a vertical upstream face for lapping the sheet piling, as previously described, reduces to a considerable degree the length of the sloping deck, on which the vertical water pressure is necessary for the stability of the dam.

10. Limitations of Timber Dams. Rock-filled timber dams have been constructed successfully to a height of about 70 ft, but very few are higher than 20 ft. The beaver type is limited by the length of the trees available. A-frame dams higher than 20 ft are seldom encountered.

B. STEEL DAMS

11. Advantages and Disadvantages of Steel Dams. The advantages of steel dams are

1. Greater speed in construction.
2. Claimed less cost.
3. Stresses more determinate.
4. Greater flexibility to resist unequal settlement without excessive leakage.
5. Unsusceptibility to frost action.
6. Easier repair of leaky joints than in hollow concrete dams, owing to modern welding processes.

Although the aversion to steel dams has been mitigated to a great extent by the excellent records of the Ash Fork and Redridge Dams, there still remain the following defects, some of which may be immaterial but which nevertheless have influenced in the past their rejection as possibilities.

1. Steel is not considered as permanent as concrete, particularly solid concrete dams.
2. Steel requires greater and more constant maintenance than concrete.
3. Steel dams, being lighter, are not as adaptable for absorbing the shock from vibrations of spilling water.
4. The types of steel dams which have been built require anchoring to the foundation, a procedure that is possible but that is not considered good practice for concrete dams.
5. Steel dams demand considerable concentration of bearing stresses, as in Fig. 6.

12. Design. The theory for the design of steel dams is the same as described for buttressed concrete dams in Chapter 19. There are two general types of gravity steel dams.

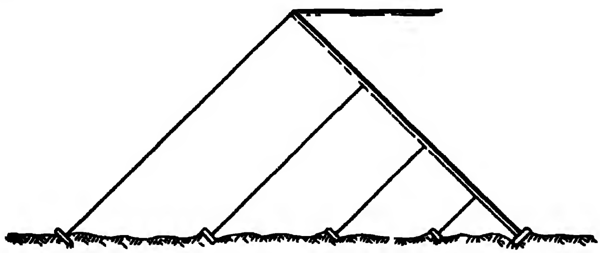


FIG. 4. Direct-strutted type of steel dam.

The direct-strutted type, shown diagrammatically in Fig. 4, is the simplest. It carries the load directly from the deck to the foundation through inclined struts.

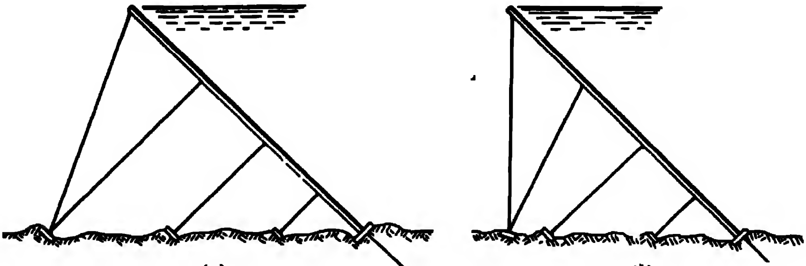
The cantilever type consists of variations of the direct-strutted type, in which the section of the bent supporting the upper part of the deck is formed into a cantilever truss, as shown in Figs. 5 and 6. This type introduces a tensile force in the deck girders that must be taken care of. This can be done in three ways.

1. The deck girder can be anchored into the foundation at the upstream toe.
2. Hovey * suggests that the tension at the upstream toe, and hence the necessary anchorage, can be reduced by flattening the slopes of the lower struts in the bent, as shown in Fig 6. That is, as the water load normal to the deck girder is transferred to the flattened strut, a component stress is induced parallel to the deck girder which will reduce the tension This adds considerably to the expense, however, because the struts must be not only longer but also of a heavier section.

* See p 26 of Ref. 5.

3. The entire bent can be framed together rigidly so that the moment of the weight of the water on the lower part of the deck may be utilized to offset the moment of the cantilever. Bainbridge says [6], however, that the cost of this bracing would be excessive.

Bainbridge states also that there is little difference in cost between the direct-strutted type of Fig. 4 and the cantilever type of Fig. 5. However, all

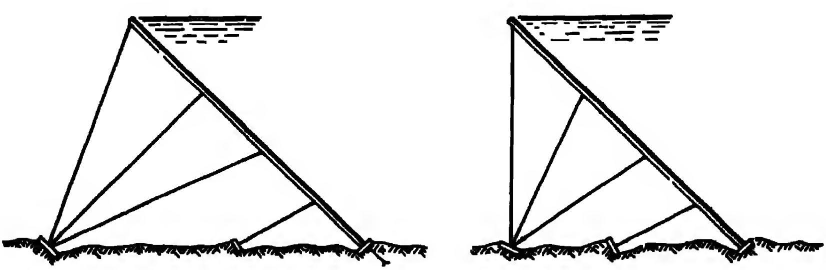


(a) (b)

FIG. 5. Cantilever type of steel dam.

three of the steel dams that have been constructed in this country and several that have been designed are of the cantilever type requiring anchorage at the upstream toe.

The cantilever type requires the lesser width of base. This was one of the considerations affecting the choice of that type for the Redridge Dam. The cantilever type, having a vertical downstream face, is more adaptable to spillway dams. However, it is not known whether this feature or the question of



(a) (b)

FIG. 6. Cantilever type of steel dam, struts flattened.

economy dictated the choice of the cantilever type for the Ash Fork Dam. The Hauser Lake Dam was of the cantilever type but also had an extensive apron to carry the overflow. Of course, where satisfactory anchorages are not obtainable, the direct-strutted type must be used.

No attempt will be made in this book to design the steel members of the dam. For this, a standard textbook on structural-steel design should be consulted. For an example of a design of a 100-ft steel dam, reference may be

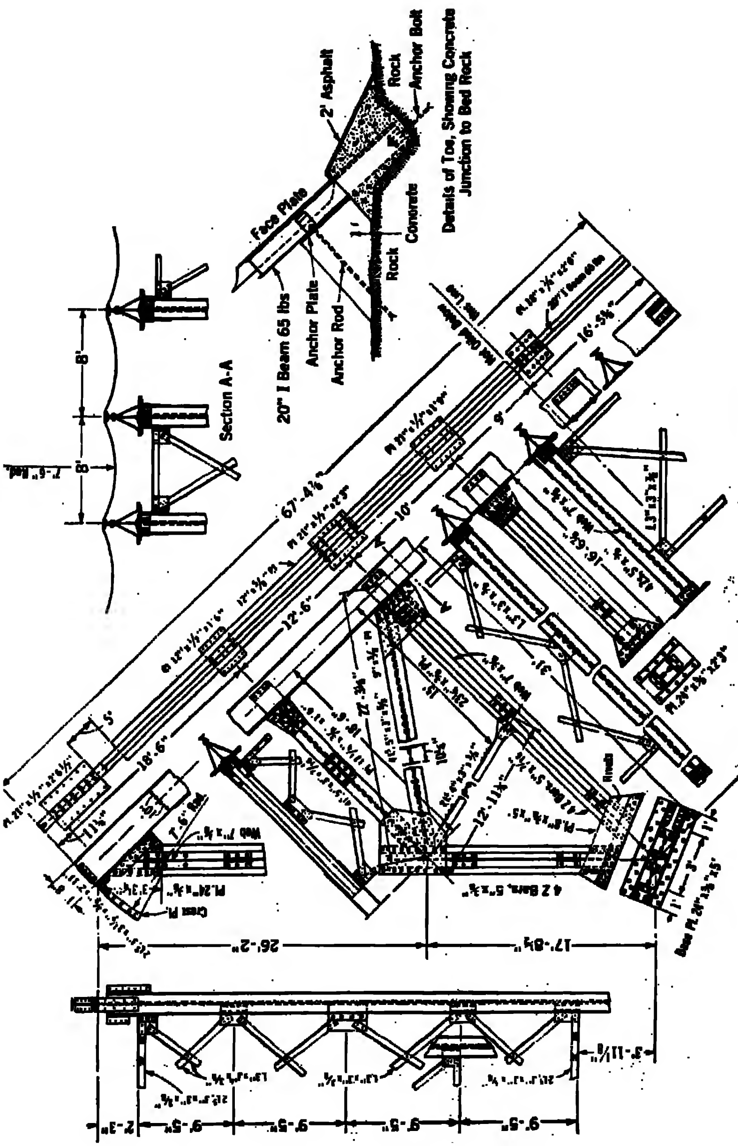


FIG. 7. Details of steel dam at Ash Fork, Ariz. (From *J. Western Soc. Engrs.*, Vol. X, 1905.)

made to a paper presented before the Western Society of Engineers by Bainbridge [6].

13. Slope of Face. As in hollow concrete dams, the upstream face of a steel dam is built on a slope in order to take advantage of the weight of the water for stability. Hovey has shown [7] that a rigid complete right-triangular frame similar to the shape illustrated in Figs. 5*b* and 6*b* would be in exact equilibrium for rotation about the downstream toe if the face were inclined at an angle of 54° 44′ with the horizontal, neglecting the weight of the structure itself. The Ash Fork and Redridge Dams, however, were both built with slopes of 45°. The Hauser Lake Dam was constructed with a 1 on 1½ slope. Bainbridge [6] says that for economy the slope should never be flatter than 45°, since as the angle decreases below 45° the weight of the face plates and deck girders is increasing faster than the cosecant of the angle, and the weight of the supporting struts is also increasing. Buttressed concrete dams are usually built with approximately a 45° slope.

14. Face Plates. The deck is made up of cylindrically curved plates, as illustrated in Fig. 7, placed concave to the water. These act practically as a suspension system [7] and will be found more efficient and economical than flat or buckle plates. These plates may be either riveted and caulked or welded to make a perfectly watertight face.

To provide for a reasonable amount of corrosion of the surface without too large a percentage reduction in thickess, a minimum thickness of plate of ⅜ in. is common practice in the design of steel pipes, where the conditions of use are similar to those in the face plates of a steel dam. A minimum thickness of ⅜ in. was adopted on the Ash Fork and Redridge Dams [6].

The tension at the connection of the plates to the deck girders will be balanced throughout the dam except at the abutments. Here anchorage must be provided to prevent bending in the web of the last deck girder.

15. Deck Girders and Bents. The deck girders supporting the face plates are most economically designed as continuous beams [7] with struts

Ash Fork Redridge

FIG. 8 Types of bracing.

spaced at increasing intervals from toe to crest. These struts, together with the girders, are arranged in a series of bents. Bracing is provided between bents, which are grouped together as shown in Fig. 8.

The spacing of the bents is a matter of economy. At Ash Fork and Redridge Dams the bents were placed 8 ft center to center.

16. Expansion and Contraction. The bays between bent groupings are left free of bracing to allow the curved plates to take up any lateral expansion

or contraction. It is essential to provide against any such movements at the abutments and foundation with good anchorage in order that these connections will not be loosened and cause leakage.

17. Foundation. Good anchorage for taking up the tension in the deck girders of the cantilever-type steel dam and for restraining temperature movements at the abutments and the foundation can be obtained by drilling into the rock and grouting anchor bars in place. Plain rods can be grouted in holes to develop their full strength. Tests in the field can be conducted to determine the proper details.

The anchorages must engage a sufficient weight of the foundation to balance the existing tension with ample factor of safety. The estimated hydrostatic uplift at the end of the anchors must be deducted to determine the effective weight of the foundation for that purpose. Direct tension in the foundation should be neglected, but each anchor can be assumed to engage a wedge-shaped portion of the foundation with central angle depending upon the nature of the rock. This is mainly a matter of judgment. Consolidation grouting, as explained in Section 3c, Chapter 25, at the top of the grouted cutoff, will materially improve anchoring in fissured foundations.

In the Redridge Dam, the steel structure is anchored to a concrete base, thus stabilizing the entire structure. Steel sheet piling driven for a cutoff may provide sufficient anchorage. This method was used for a section of the Hauser Lake Dam [3].

It is essential to make all connections to the abutments and foundations watertight. If there is to be a concrete cutoff wall, the face plates should be buried in the concrete. On solid rock it is best to build a low concrete wall, well anchored to the rock, and imbed the face plates in the concrete. The plates may also be welded to a steel sheet pile cutoff. In order to make the connections at the foundation, the curved face plates are replaced by flat steel plates. These are connected to the curved plates by means of a segment cut to fit and welded in place. The flat plates may then be bent down into the concrete wall or against the steel sheet piling.

The remainder of the foundation problem is no different from that described for concrete dams. However, in horizontally stratified rock, the horizontal shearing resistance should be assumed to be limited to the width of the base of the individual struts unless such bases are deeply imbedded.

There is, of course, no uplift on the bases of the struts. However, the problem of eliminating or balancing uplift on horizontally stratified planes below the base is no different from that for a solid concrete dam.

18. Durability and Painting. The Ash Fork and Redridge Dams were built in 1898 and 1901, respectively. Up until 1935 the Ash Fork Dam had been repainted on an average of each 7 to 9 years and the Redridge Dam had been repainted only once [5]. Both dams are claimed to be in excellent condition. It is evident that there will be little question of the durability of the steel provided it is adequately protected by paint and that paint is properly maintained. It is entirely possible that, in the future, with the develop-

ment of economical noncorrosive steels, the need for vigilant maintenance of a protective coating of paint will be eliminated.

19. Quantities and Costs. Bainbridge has computed that the quantities in steel dams average about 12,500 lb per lin ft of dam for a dam 100 ft high [6]. He states that the Ash Fork Dam, 42 ft high, averaged about 2000 lb per lin ft of dam for face plates and deck girders and about 1500 lb per lin ft of dam for struts and bracing.

Published estimates of steel dams indicate a lower cost than that of any type of concrete gravity dam for the same site [6, 7, 8].

Estimates by Stanley [8] indicate that the higher the dam the greater the advantage of steel with respect to cost. This fact has also been noted [6, 7] by Bainbridge and Hovey.

20. Steel Arch Dams. Arch dams composed of steel are feasible, but to the author's knowledge they have never been built. A dam of this type has been proposed by Stanley [8].

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CHAPTER 25

PREPARATION AND PROTECTION OF FOUNDATIONS FOR DAMS

1. Scope. With the exception of foundations for dams, the problem of adequate foundations for the many structures appurtenant to hydroelectric developments is no different from that of structures for other uses. The foundations for dams deserve special discussion. The subject has been adequately treated in books on dams. This chapter will cover those features that affect the choice of type of dam and the general design and that provide information, as far as possible, for estimates of work involved.

2. General Considerations. The term "foundation" as used herein means all that part of the area under and adjacent to the dam which in any way will be affected by loading, scour, or leakage. A good foundation is of ample strength to withstand the weight of the structure and to prevent sliding. It must be tight enough to prevent excessive leakage; uplift must be reduced as much as possible; and discharge from the overflow or outlets must not damage it.

Considerable preparation is always necessary in order to provide the requisites of a good foundation. It is probable that more than 90% of all failures of masonry dams have been caused by faulty foundations.

Shallow overburden is always removed to obtain a rock foundation for a dam. However, where the depth of overburden is excessive, dams up to about 65 feet in height have frequently been built on earth. As the cost of a dam on earth is considerably greater than one on rock, the choice of which to use when the rock is neither very deep nor very shallow becomes an economic problem.

3. Treatment of Rock Foundations. (*a*) *Final Surface of Rock Foundations.* Surface rock is usually so badly weathered as to be unsuitable for the support of a dam. Excavation to considerable depths is sometimes necessary before rock of an acceptable nature is uncovered. Without extensive subsurface explorations, it is sometimes impossible to determine the depth of suitable foundations. However, the services of a competent geologist and the examination of rock cuts in the vicinity will be helpful.

There should be as much resistance to sliding below the surface of the foundation as at the surface. If, therefore, an otherwise good foundation of firm rock contains loose horizontal or nearly horizontal strata on which there is danger of sliding, the foundation should be treated as explained in Section 19, Chapter 17.

For buttressed dams on uncemented shales, with weak, nearly horizontal bedding planes or other types of weak foundations, a deep heel trench, below the upstream side of the dam, excavated by line drilling and broaching, is provided to be filled with reinforced concrete securely anchored to the dam, as in Fig. 1.

In order that the masonry of the dam shall have the maximum possible bond with the foundation, it is necessary that the final rock surface be abso-

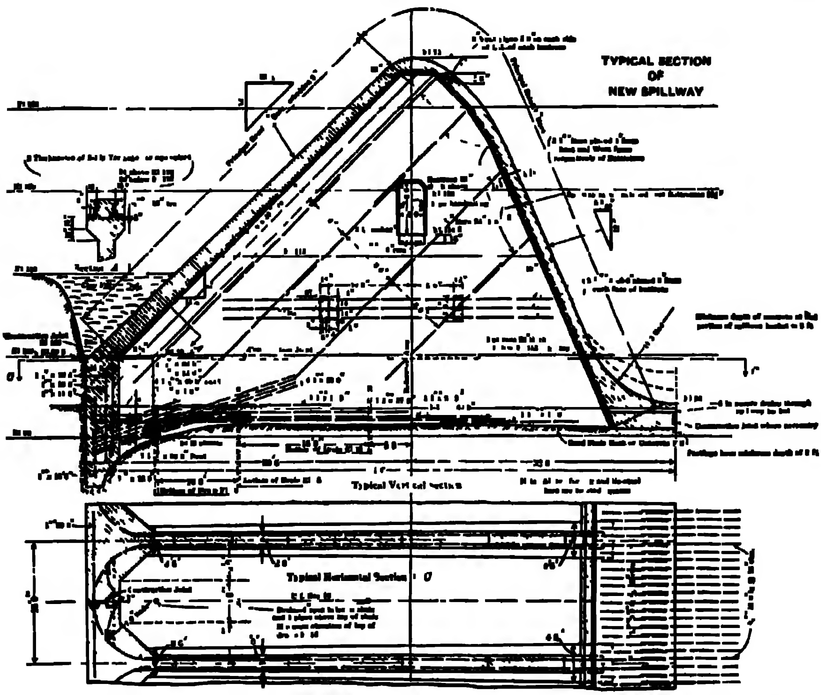

Fig. 1. Stony River Dam (F. W. Scheidenhelm, *Trans. A.S.C.E.*, 1917, p. 907).

lutely clean. The thinnest film of dirt of any sort on the surface of the rock is capable of destroying adequate bond and may defeat the entire purpose of the cleansing operations.

There should be no large overhanging faces in the rock foundation, and vertical faces should be avoided, except where they coincide with vertical building joints, as they tend to cause shrinkage cracks and excessive shearing stresses.

Where rock, such as uncemented shale, tends to disintegrate when exposed, the final trimming should not be done until just before the concrete is to be placed; otherwise the rock might dry out and, when again saturated by the water from the concrete, might form a layer of mud between the foundation and the dam, offering no bond and little resistance to sliding.

In extreme cases, where the rock disintegrates by slaking quite rapidly, the final foundation, as soon as uncovered, should be coated immediately with a bituminous or asphaltic waterproofing material. This procedure not only is of greatest importance in soft shales, but also adds to the strength of the bond between the shales and the concrete.

(*b*) *Reduction of Leakage.* Some seepage or leakage through rock foundations is to be expected, as bedrock is seldom, if ever, entirely unbroken. The chief objections to such leakage are its effect on uplift, the possibility of its washing out intermediate layers of clay or sand, waste of water, and objectionable appearance.

To confine the leakage to a reasonable quantity, it is necessary, with poor foundations, to provide a cutoff or artificial impervious curtain barrier under the heel of the dam. For rock foundations there are two general types of cutoffs: first, a trench filled with concrete, and second, curtain grouting consisting of holes drilled at frequent intervals and grouted under pressure.

The first type is much to be preferred if it can be constructed at reasonable cost. Before the use of grouted cutoffs became common, the concrete cutoff was sometimes carried in exceptional cases to depths of 50 ft or more. Although its advantages are obvious, it has been found that the grouted curtain cutoff is usually sufficiently effective and costs much less.

Most high masonry dams are provided with a grout curtain at the upstream end of the base to reduce leakage, in conjunction with drainage holes immediately downstream from the curtain to relieve the foundation of uplift pressures, as described in Section 7 of Chapter 17 and later in this chapter. For low dams on good rock foundations, such grouting and drainage are not necessary, as indicated in Sections 7 and 18 of Chapter 17.

(*c*) Grouting Rock Foundations. Grouting of rock foundations serves both for tightening to prevent leakage and reduce uplift and for consolidating seamy and broken foundations to make them stronger. Cement grout usually consists of a mixture of neat cement and water. Thin grout travels farther than thick grout. Hence, it is customary to start with a thin mixture of 1 part of cement to 5 or 6 parts of water and gradually thicken the grout as the hole tightens. This procedure not only reaches remoter seams but also offers a minimum of disturbance of the natural formation. A mixture of 1 part of cement to about 2 parts of water is frequently adopted for the final grouting, but where the rock is loose, a mixture of 1 part of cement to 0.6 part of water has been used. Where there is difficulty in sealing a seam, sawdust or shavings may be added to thick grout.

Hot asphalt grouting has been found successful [6] for cavities containing running water.

There are several types of grout curtains to prevent leakage, the choice depending upon the nature of the foundation. Each type consists of one or several lines of holes. More than one line of holes is not necessary unless rather high pressures are required for the deep holes, in which case several other parallel lines of shallow holes may be required to consolidate the surface

over a wider area. The difference between the methods is in the relative depth and sequence of grouting the different holes in the curtain.

Fault gouges, stiff clay, or mudstone in thin layers are not particularly objectionable, as far as seepage is concerned, but seams containing mud, silt, or sand may wash out after the reservoir is filled. These materials, therefore, should be cleaned out of the seams before grouting. For large solution channels it is best to provide access, clean out and concrete, grouting being applied for a final seal.

The upper end of all holes to be grouted should be provided with threaded pipes, which can be connected to the grouting machine. These pipes must be anchored or weighted to prevent a blowout during the process of grouting. This is sometimes done by cementing the connecting pipes into the drilled holes, or by placing them in a concrete cutoff carried far enough into the rock to provide ample grip. The drilling, in the latter case, is usually done through the pipes.

Pressures used for grouting must be carefully limited to those that will not lift or otherwise move any part of the foundation or adjacent structures. On the other hand, the pressures must be as great as are allowable for speedy work and the largest possible coverage.

When difficulty is experienced in adequately grouting the surface layers without lifting them and when it is feared that high-pressure deep grouting may lift the foundation, curtain grouting is done from a gallery after the dam is built or partly built. Machines can be obtained to drill vertical or inclined holes in a 5- by 8-ft gallery.

For curtain grouting, the required depth of the deepest holes naturally depends on the nature of the rock in the foundation. The general rule that the deepest grouting should extend below rock surface a distance equal to one fourth of the hydrostatic head above rock surface is too inaccurate for serious consideration. The only accurate determination of required depth is a water-pressure test to determine leakage in segregated zones at different elevations. This test for each hole consists of pumping clear water into the hole at fixed pressure and measuring the discharge.

As more fully described in Chapter 7, most unweathered cemented rocks possess sufficient strength to support dams of usual height. However, special consideration should be given to rocks in which seams or faults and weathered or crushed zones have resulted in separated or partly separated foundation blocks that might move slightly as a whole under the load of the dam. Such foundations have been successfully treated by consolidation grouting, which procedure, including the washing out of seams, is very similar to that described for curtain grouting, but the required depth is seldom, if ever, as great. It is essential that all seams be thoroughly washed out before consolidation grouting begins.

Narrow seams and faults frequently can be washed out and grouted. For wide seams, the gouge, weathered or broken rock, or other material filling them can be excavated and the seams refilled with concrete. When such

defective material lies in a nearly horizontal plane below the surface of the completed excavation, it is sometimes more economical to reach it by a vertical shaft or a man-size drill hole, clean out the seam in drifts, and fill it with concrete rather than to excavate the firm rock above it. Cavernous rock and solution channels can also be treated in this manner.

For low dams, small areas of relatively weak rock are sometimes left in place on the assumption that the dam will span over them. In several cases, vertical transverse faults of considerable size have been cleaned out and filled with concrete for a depth only sufficient to provide an arch to span the opening, care being taken of course that the excavating or grouting extends far enough to obtain a tight cutoff at the upstream side.

It is impossible to indicate by any rule the necessary extent or probable cost of grouting. Estimates must be based on experience, on the nature of the foundation as indicated by preliminary investigation, and on experience with grouting requirements for dams on similar foundations.

(*d*) *Draining Rock Foundations.* In order to relieve hydrostatic pressure and reduce uplift on the base of the dam and in seams in the foundation, it is frequently advisable in seamy rock, and always for high dams, to drill a line of holes downstream from the previously placed grout curtain to carry away any seepage water which may pass the curtain. (See Section 7 of Chapter 17 and Fig. 18*b* of Chapter 17.) The drainage holes are connected to a drainage gallery or other means provided to carry the seepage to tailwater. They should not be drilled until all grouting operations are completed and should be far enough downstream from the grout curtain to be sure of intersecting only open seams.

There is no fixed rule for determining the size, spacing, and depth of drainage holes. Experiments on the relative effectiveness of different combinations are badly needed. The holes vary from 2 to 6 in. in diameter and from 5 to 20 ft from center to center. Their depth depends on the character of the rock, but a depth equal to one quarter to one half the width of the base of the dam is a fair average.

(*e*) *Toe Protection.* Protection of the rock at the toe of spillway dams is frequently needed, particularly if the rock has horizontal stratification. Methods of reducing the high velocity of spilling water to a velocity that will not erode the rock are described in Section 5.

4. Treatment of Earth Foundations. (*a*) *Adaptability of Earth Foundations.* Concrete dams on earth foundations are numerous; but their use in this country has been limited for the most part to structures not more than about 65 ft high for good earth foundations and 30 ft high for less resistant earth. This limitation in height may be attributed to the fact that the treatment of earth foundations, to prevent erosion and excessive seepage, is far more expensive than that necessary for rock foundations. In fact, the cost of foundation treatment for dams on earth is often the major part of the total cost of the structure. Consequently, for moderate and

high dams it will be found best to adopt another type of structure or to change the site. There have been few precedents for dams higher than noted above, although structurally there would seem to be no reason for a limit to the height, provided that sufficient funds are available to meet the unusual expense.

The preparation of the foundation for a dam on earth must be made with five objectives in view: (*a*) to provide ample bearing strength; (*b*) to prevent sliding; (*c*) to prevent excessive seepage under the dam; (*d*) to prevent piping; (*e*) to prevent scouring by the water passing over the dam.

(*b*) *Bearing Strength of Earth Foundations.* The allowed loads on earth foundations are discussed in Section 20 of Chapter 17. It is essential that there be no excessive settlement in the structure. Unequal settlement is particularly objectionable, as the tightness of the structure is dependent on the absence of settlement cracks.

For hollow dams on earth, the footings are usually spread to reduce bearing stresses. The weight of the hollow Mathis Dike Dam was distributed over the foundation by a mattress covering the entire base, as indicated in Fig. 5, Chapter 19. The weight of solid dams can be distributed through the aprons, which are often reinforced for that purpose. Sometimes bearing piles are required.

(*c*) *Bearing Piles.* Many types of wood, concrete, or steel bearing piles have been used for excessively weak foundations, the type depending on the length required and the nature of the foundation materials. The subject of bearing piles is too lengthy for proper treatment here, and the reader is referred to the latest reports of the Committee on Bearing Value of Pile Foundations of the Waterways Division and the Committee on Foundations of the Soil Mechanics Division of the American Society of Civil Engineers.

Any type of piling under concrete dams, including cutoff piling and bearing piling, is conducive to roofing. Where such piling is used, the possibility of roofing should be guarded against by providing filter drains to prevent piping, as indicated in Fig. 2, by an upstream blanket to provide sufficient length of path of percolation, or by a combination of these expedients.

Bearing piles should be avoided if a spread base will serve the purpose. Foundations with a dry weight of less than 100 lb per cu ft should be looked upon with grave suspicion. Such loose foundation material generally should be removed.

(*d*) *Sliding on Earth Foundations.* The dam must, of course, be prevented from sliding, as explained elsewhere in this book. Where the coefficient of friction of concrete on the earth is insufficient, vertical bearing piles, if used, can be counted on to resist the water pressure, although a slight horizontal deflection must be expected. Considerable expenditures have sometimes been made for tests on full-size piles to determine such deflections. However, for correct results such tests must be made on pile clusters and not on individual piles.

In some dams (see Fig. 2) battered piles have been used effectively to prevent incipient sliding. In the Mathis Dike Dam (Fig. 5, Chapter 19) the longitudinal ribs at the toe and the middle of the base are intended to assist in preventing the dam from sliding.

In some cases increased resistance to sliding has been obtained by thoroughly anchoring the dam to a deep cutoff wall at the heel of the structure. Figure 1 shows such a cutoff for a section of the Stony River Dam where the dam rests on shale. The same method was used also to prevent sliding

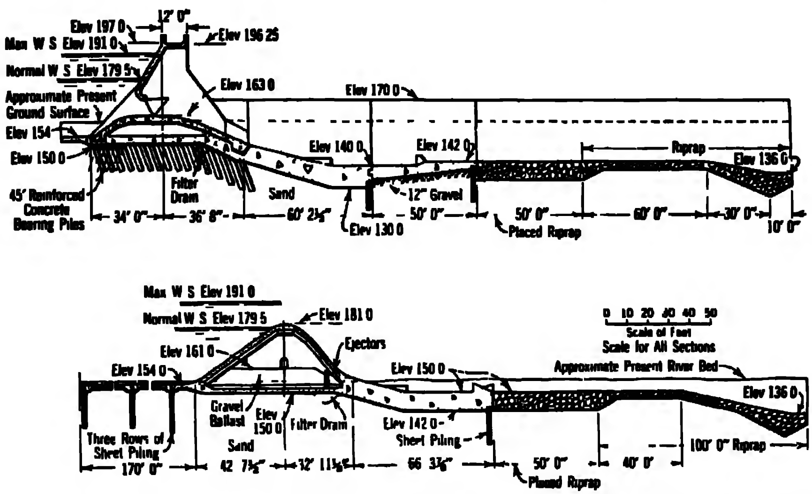

FIG. 2. Imperial Dam, Colorado River, Ariz (*Dams and Control Works*, 2nd edition, U. S. Bureau of Reclamation, 1938)

where it rested on a clay foundation. The cutoff must, of course, be well tied to the main structure, as shown.

(*e*) *The Flow Net*. The flow net is a diagrammatic representation of the lines of percolation and lines of equal potential in a porous medium such as earth subject to a head of water. Figure 3*a* shows a typical flow net for a homogeneous, isotropic foundation without drains or cutoffs. *A–B* is the length of the impervious elements of the dam. The solid lines are the lines of percolation, or flow lines, and the dotted lines are lines of equal potential In the true flow net, all the areas bounded by any pair of flow lines and any pair of equipotential lines are homologous, i.e., have the same ratio of width to length.

The potential at any point in the foundation is the elevation to which water would rise in a piezometer at that point, as shown in the figure. For homogeneous, isotropic materials, the flow lines are normal to the equipotential lines. The friction loss of seepage along any flow line is equal to the head, *h*, on the dam. The head lost in friction between any two lines of equal potential is equal to the difference in their potentials.

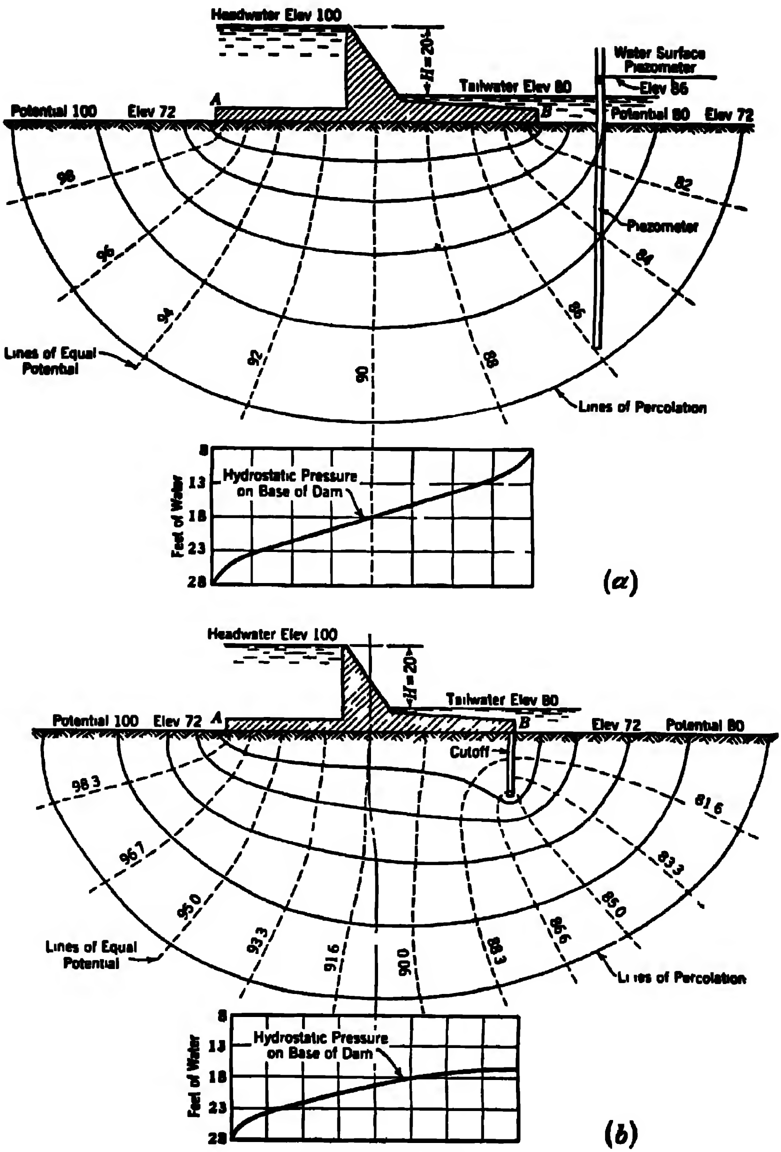

FIG. 3. Flow net.

The longer the path required for the flow lines, the farther apart will be the lines of equal potential, the smaller the friction loss per linear foot, the slower the velocity, and the less the seepage per square foot.

The hydrostatic pressure, in feet of water, at any point on the base of the dam can be determined from the flow net simply by subtracting the elevation of the base from the potential at that point, as shown in Fig. 3a. One hundred per cent of the hydrostatic pressure is always assumed for uplift for dams on earth.

Should the permeability at all points in the foundation be increased, the amount of seepage and velocity of flow would be increased in direct proportion but the flow net and amount of uplift on the dam would not change.

The flow net can be determined analytically for simple conditions; but, where there are one or more cutoffs, strata of variable permeability, lenses of different permeability, drains, and other complications, it can be determined most conveniently by the electrical-analogy model test. The flow net should be determined for all important dams when a tight cutoff to impervious material is not possible.

The "shortest path of percolation" is the shortest line of percolation as shown by the flow net. Bligh's and Lane's "line of creep," discussed more thoroughly in Section 4i, is the line of contact of the base of the dam and cutoffs with the foundation. Should the line of creep be much more pervious than the rest of the foundation, the flow net for a homogeneous foundation would not apply as the percolation would tend to follow the line of creep. The term "path of percolation" will be used in a general sense, since those features which tend to influence the path will apply both to homogeneous foundations and to the line-of-creep theory.

Other things being equal, a long path of percolation reduces seepage and the possibility of piping. The effect of the length of the path of percolation on uplift will be explained in Section 4h.

The desired length of the path of percolation can be obtained by an upstream apron, a downstream apron, one or more cutoffs, or a combination of all these, as later explained in more detail. Many types and combinations have been proposed and built. Modern practice in this respect will be discussed later.

(f) Seepage through Earth Foundations. A small amount of seepage through earth foundations for dams is to be expected. Excessive seepage is objectionable, not only on account of waste of impounded water, but principally because of danger of piping. Seepage is reduced to best advantage by a watertight cutoff to rock or other impermeable stratum. It can be reduced also by increasing the length of path of percolation, as previously explained.

In silt-laden streams deposits in the reservoir bottom filtered out of turbid water will often seal the pores quickly and thus reduce seepage. The Granite Reef Dam, shown in Fig. 4, is founded on gravel and boulders. The length of the path of percolation was only about three times the head on the dam.

The considerable leakage when the dam was first used was soon stopped by the large quantity of silt carried by the river.

A cutoff, under a dam, which carries only part way to an impervious stratum is not efficient. Figure 5, plotted from the results of research tests

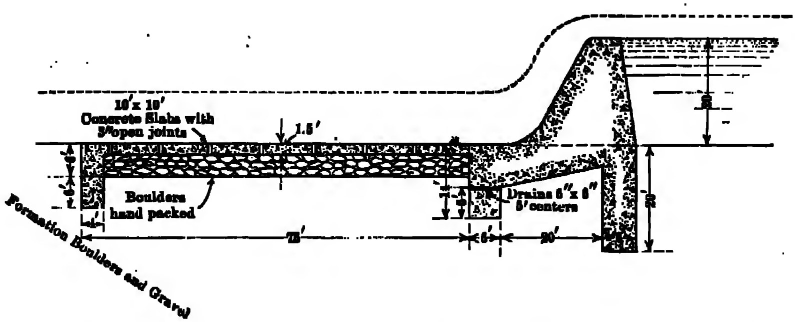

FIG. 4. Granite Reef Dam, Salt River, Ariz. (*The Design and Construction of Dams*, Wegmann.)

by Creager and also tests by Turnbull, for the Kingsley Dam in Nebraska, shows the relation between seepage and depth of cutoff. Those by Creager were for a ratio of base width of dam to depth of pervious material of 1.4; the corresponding ratio for the Kingsley Dam was 14.

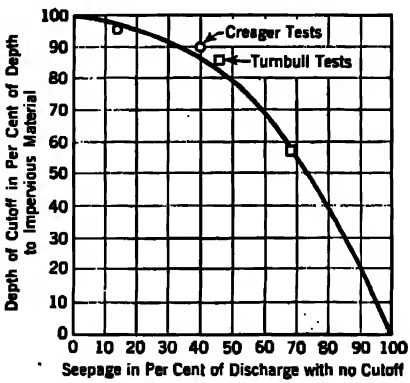

FIG. 5. Relation between seepage and depth of cutoff.

Figure 5 shows that, for a cutoff occupying 90% of the distance to the impervious material, the seepage will still be 35% of the seepage for no cutoff.

If the permeability of the foundation, K, is known, the amount of seepage past any length of dam can be computed from the flow net. Casagrande has

shown* that the seepage loss in cubic feet per second may be found by the following equation, if the flow net is formed of squares.

$$Q = \frac{LKHs}{60} \qquad [1]$$

when Q = discharge in cubic foot per second;
L = length of the dam considered, in feet;
K = coefficient of permeability of the material in cubic foot per minute per square foot of area;
H = head on the dam in feet;
s = ratio of the number of squares between any two neighboring equipotential lines and the number of squares between any two neighboring lines of percolation.

For example, the coefficient of permeability, K, is the discharge in cubic feet per minute per square foot of the foundation material under unity gradient, i.e., when the friction loss per foot of travel is unity.

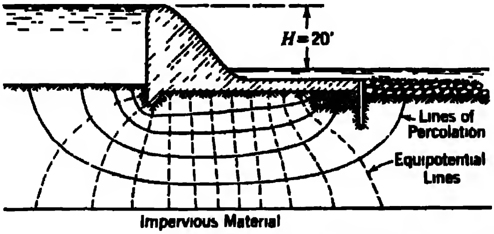

FIG. 6. Example of flow net for seepage calculation.

In Fig. 6 there are five squares between any two neighboring equipotential lines and twelve squares between any two neighboring lines of percolation. Therefore,

$$s = \tfrac{5}{12} = 0.416$$

Assume that $K = 50 \times 10^{-4}$ cu cm per sec per sq cm,†

$$K = \frac{50 \times 60}{10{,}000 \times 30.48} = 0.00984 \text{ cu ft per min per sq ft}$$

and

$$H = 20 \text{ ft} \quad \text{and} \quad L = 500 \text{ ft}$$

Then the expected seepage would be

$$Q = \frac{500 \times 0.0098 \times 20 \times 0.416}{60} = 0.68 \text{ cu ft per sec}$$

* A. Casagrande (after A. F. Samsioe), *Trans. A.S.C.E.*, 1935, p. 1289.
† Usual laboratory nomenclature.

(*g*) *Piping.* Piping may be defined as the movement of material from the foundation by the velocity of the seeping water as it issues from the soil below the dam. Under such conditions the soil becomes supersaturated, quick, and incapable of supporting any load. The dam should therefore be investigated with respect to that condition.

The closer the lines of equipotential, the higher the seepage velocity and the greater the loss of potential per linear foot. For the simple case of Fig. 3*a*, used here for explanatory purposes but not as a recommended design, it will be noted that the lines of equal potential are closer at *A* and *B* than elsewhere. This is true wherever there is an abrupt corner about which the water must flow. Therefore, at such places the rate of loss of potential and velocity of seepage is relatively very great. Theoretically, it is infinite exactly at the point of turn.

This is a matter of no concern at the entrance surface or in the foundation, where this condition cannot move the material, as the material has no place to go. However, it is conducive to piping where the seepage emerges below the dam.

In Fig. 3*a* the high velocity at *B* would assuredly cause piping.

These theoretical conditions would be improved, as far as piping is concerned, by the addition of a cutoff at the downstream end, as shown in Fig. 3*b*, which would transfer the high seepage velocity to the bottom of the cutoff.

However, note that for Fig. 3*b* the downstream cutoff has increased considerably the uplift pressure on the downstream apron and, for that reason, is objectionable. On the other hand, where controlled drainage (later recommended) is not used, some depth of cutoff at the toe, to reduce surface exit velocity, is absolutely necessary.

The theoretical conditions affecting piping are discussed in books on dams. Piping may be prevented by providing a sufficiently long path of percolation under the dam, downstream piling, and drains, as subsequently described.

(*h*) *Uplift.* As indicated in Chapter 17, the uplift pressure on the base of the dam must be considered in the determination of the stability of that structure. The determination of the amount of uplift from the flow net is explained in Section 4*c*. If an analogy model test is not made, the uplift to be taken care of should be determined from a comparison of the flow nets of similar designs. An ample factor of safety should be used where the foundation has not been thoroughly explored and lenses of different permeability or other heterogeneous conditions may exist.

Uplift must be considered not only at the base of the dam and apron but also at deeper points in the foundation. For instance, a dam or apron above an adequate filter drain would have no uplift on its base, but there would still be hydrostatic pressure at points a few feet below the base. In extreme cases, as mentioned before, this hydrostatic pressure might be capable of lifting the saturated weight of earth above it and the weight of the apron

and water above the apron. In other words, all filter drains must be adequately weighted. Any uplift on the apron below the dam must be balanced by the weight of the apron and water above it. However, the force of the spilling water may reduce the depth of water over the apron as explained in Section 5.

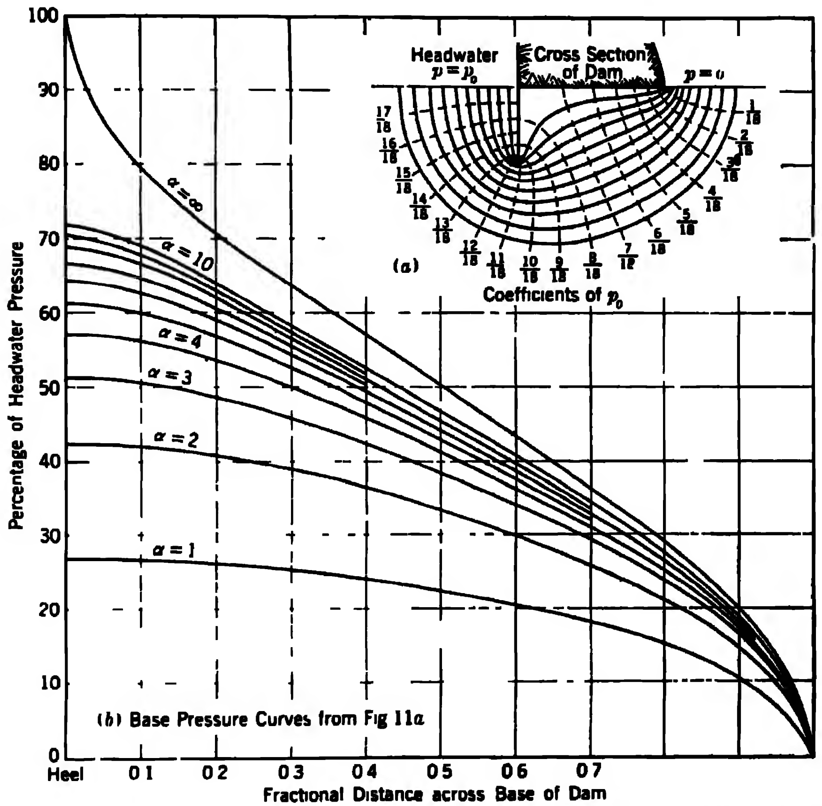

FIG. 7. Flow net for heel cutoff. (L. F. Harza, *Trans. A S C.E.*, 1935, p. 1367.)

A downstream apron to increase the length of the path of percolation will increase uplift on the dam. A cutoff at the upstream end of the dam or an upstream apron will reduce uplift. Weaver [12] has determined mathematically the data from which Fig. 7 was drawn, to indicate the effect of a single upstream cutoff on uplift for a dam on a pervious foundation of infinite depth without drains. In Fig. 7, α is the ratio of the base width of dam and aprons to the depth of the cutoff. Values of α = infinity correspond to no cutoff and agree with Fig. 3*a*.

A lens of relatively pervious material downstream from the heel of the dam reduces uplift. Such a lens upstream from the heel increases uplift.

The thickness of the downstream apron, subject to uplift, can be reduced if anchored to piles driven into the foundation.

(i) *Line of Creep.* Bligh's line-of-creep theory revised by Lane [8] is based on the theory that resistance to percolation along the "line of creep," i.e., along the line of contact of the dam and cutoffs with the foundations, may be less than directly through the foundation materials on account of the difficulty of securing an intimate contact. Thus, according to this premise, the flow will concentrate along the line of creep, causing higher velocities at the exit than would be indicated by a flow net constructed for a homogeneous foundation. This loosening of the contact between the dam and the foundation, caused by unequal settlement and general settlement where bearing piles are used, is called "roofing."

Because the exact nature of roofing and disturbance of the contact of cutoffs with the foundation is indeterminate, Bligh recommended a minimum "creep ratio" of length of line of creep to head on the dam, dependent on the nature of the materials in the foundation. His recommended creep ratio was based on the observation of the success or failure of a number of dams.

Adopting the same general theory, Lane, after a study of more dams of this type, recommends a weighted line of creep, in which horizontal contacts with the foundation and slopes flatter than 45 degrees, being less likely to have intimate contact, are given only one third the value of steeper and vertical contacts. That is, his line of creep is the summation of all the steep and vertical contacts plus one third the sum of all the flatter and horizontal lines of contact between headwater and tailwater following along the contact surface of the base of the dam and cutoffs.

Should the distance between the bottoms of two cutoffs be less than one half the weighted creep distance between them, twice the distance between them should be used instead of the actual line of creep between them.

Lane's recommended weighted creep ratios, i.e., the ratio of weighted creep distance to head, are given in the first column of Table 1. They are subject to certain modifications for special conditions.

The authors' opinions, fortified by the opinions of the discussors of Lane's paper and by Lane's closure of the discussion, are indicated in Table 1 and explained as follows:

Case a. Although the authors will later recommend provisions for filter drains and downstream cutoffs wherever possible and the use of flow net analyses for important dams, it is recommended that, where these provisions are not made, Lane's weighted creep ratios be used, as indicated in Table 1 (Case *a*).

Case b. Where drains are properly provided but no flow net analyses are made, use 80% of Lane's weighted creep ratios.

Case c. Where both drains and flow net analyses are employed, adopt 70% of Lane's weighted creep ratios even though the flow net analysis may indicate that a smaller ratio would be safe.

TABLE 1

RECOMMENDED WEIGHTED CREEP RATIOS

Material	Case *a* (Lane) 100%	Case *b* 80%	Case *c* 70%
Very fine sand or silt	8.5	6.8	6.0
Fine sand	7.0	5.6	4.9
Medium sand	6.0	4.8	4.2
Coarse sand	5.0	4.0	3.5
Fine gravel	4.0	3.2	2.8
Medium gravel	3.5	2.8	2.5
Coarse gravel, including cobbles	3.0	2.4	2.1
Boulders with some cobbles and gravel	2.5	2.0	1.8
Soft clay	3.0	2.4	2.1
Medium clay	2.0	1.6	1.5
Hard clay	1.8	1.5	1.5
Very hard clay or hardpan	1.6	1.5	1.5

For Cases *b* and *c*, however, the weighted creep ratio should not be less than 1.5 under any circumstances.

However, the base of dams and aprons supported on bearing piles should be given a weighted creep of zero if a filter drain is not provided as shown in Fig. 2.

(*j*) *Recommended Design for Earth Foundations.* There have been many combinations of appurtenances for foundation protection, consisting of one or more upstream cutoffs, upstream aprons or blankets, downstream aprons, one or more lines of downstream sheet piling, and drains of various kinds at diverse places. An indication of the extreme variance in design is shown by examples of foundation treatment for over 150 dams on earth, contained in a supplement by Lane [9]. This reference also contains an excellent bibliography.

So much depends on the conditions at the site that no general standard is in use. However, Fig. 8 can serve as a basis, variations from which must be made to suit local conditions. It consists of an upstream apron, a blanket with filter (described later), an upstream cutoff, a main filter drain, a downstream apron above a filter drain, and a downstream row of sheet piling. The downstream row of sheet piling is used to prevent damage if the riprap washes out and to reduce the velocity of residual seepage, as previously explained.

Several cutoffs were used at the Imperial Dam (Fig. 2). However, Creager has found that a single cutoff at the upper end of the upstream apron has proved sufficiently effective with, perhaps, an auxiliary cutoff at the upstream end of the base of the dam to reduce slightly more the uplift on the dam and to serve as a safeguard in the event of failure of the apron. The best arrangement can be obtained only with a flow net from an analogy test.

A filter drain was located at the downstream toe of the Imperial Dam. It was probably proved by model tests to be adequate, since no filter drains are located under the apron. The weighted creep ratio is 8.0 for a sand foundation. Many safe dams, under similar conditions, have been built with much lower ratios.

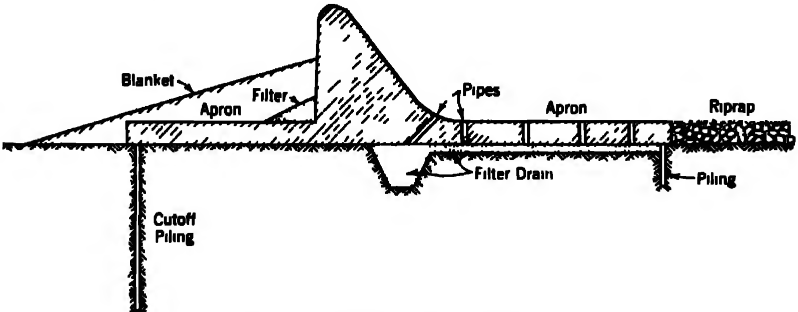

FIG. 8. Diagrammatic example.

The Granite Reef Dam (Fig. 4) rests on a foundation of gravel and boulders. The weighted creep ratio when the dam was first built was 2.8, and no piping occurred. When the dam was first used, however, considerable water passed under the cutoff and out through the drains on the apron. This explains the occasional necessity for increasing the weighted creep

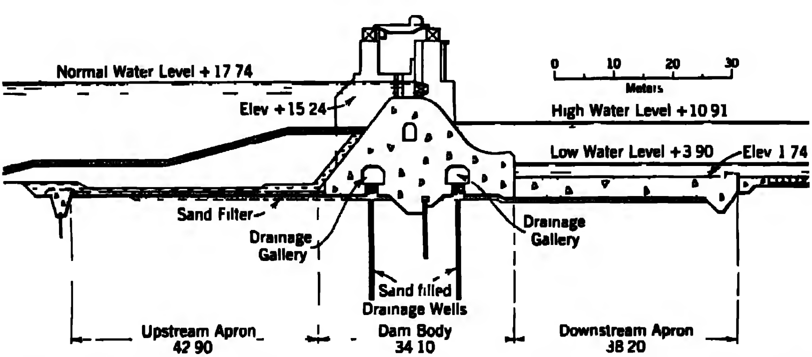

FIG. 9. Svirstroy Dam, U.S.S.R. (*Proc. Int. Conf. Soil Mech.*, Cambridge, Mass., 1936).

ratios of Table 1 to limit waste of water, as previously explained. For this dam, the flow was soon stopped by silt deposited on the floor of the reservoir.

The Svirstroy Dam, U.S.S.R. (Fig. 9), has a cutoff at the upper end of the upstream apron. Since the foundation was clay, this cutoff supplied sufficient length of path of percolation to eliminate excessive leakage. Extending the filter drain under the upstream apron reduced uplift and therefore

caused the greatest net downward water pressure on the apron. Since the apron was tied into the dam proper, this pressure assisted in preventing sliding.

The Cochiti Dam (Fig. 10) rests on sand, gravel, and cobbles. It has a weighted creep ratio of 5.5, a conservative value. A "selected-gravel" drain was placed under the dam where shown.

(*k*) *Upstream Apron.* The upstream apron is used, of course, to increase the length of the path of percolation. It is better than a downstream apron for that purpose, since there is no unbalanced uplift under it to be balanced by weight of concrete.

It may be made of reinforced concrete, or it may be simply an impervious earth blanket. An opening of the joint between the apron and the dam, due to unequal settlement, must be guarded against.

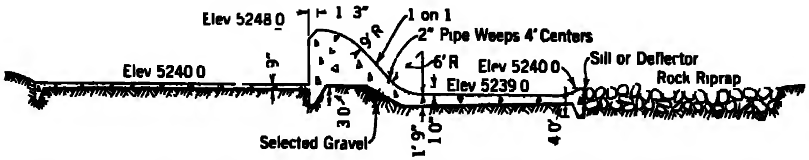

FIG. 10 Cochiti Dam New Mexico (*Civil Eng*, January 1933, p 12)

Figure 8 shows a filter at the junction of the apron and the dam and an impervious blanket above the apron. Should a crack occur at that place, the filter will prevent the blanket from washing through. The blanket may require riprapping for protection from high velocity in the pond or wave action if the pond is drawn down.

At the Svirstroy Dam (Fig 9), in order to obviate the possibility of leakage at the junction of the upstream apron and the dam due to possible cracks caused by unequal settlement, the junction was composed of flexible asphalt concrete on top of which was placed a layer of pure asphalt covered by earth fill. The junctions between the apron and the cutoff walls as well as the contraction joints in the apron were similarly treated.

In the dam shown in Fig. 10, a filter drain and blanket would have been advisable above the junction of the upstream apron and the dam, as indicated in Fig. 8, had the designers feared the possibility of a crack in the apron occurring at that place.

(*l*) *Cutoffs for Dams on Earth.* Section 4*j* discusses the general arrangement of cutoffs. Obviously, wherever practicable, the cutoff should be carried to impervious materials. The cutoff may consist of a concrete diaphragm, interlocking-steel sheet piling, or tongue-and-grooved wood sheet piling.

The only situation in which a concrete cutoff should be used is that in which impervious materials can be reached and boulders prevent the use of sheet piling. When sheeting is employed for excavation, the sides of a concrete cutoff trench are disturbed and may allow ready seepage. This reduces the efficiency of such a cutoff unless the cutoff is carried to impervious

material. Also, an upstream apron of sufficient length to be as effective as a concrete cutoff going only part way to impervious material generally proves more economical. The width of the concrete cutoff may be as small as excavation will permit. It needs no reinforcement except if tied into the dam to help prevent sliding.

Steel sheet piling has been gaining favor for dams on earth. Where the driving is easy, the depth is not great, and particularly where jetting is feasible, light shallow arch piles may be used. At the Fort Peck Dam piles of this type of 23 and 28 lb were driven 150 ft with the aid of jets. The lighter type was employed under similar conditions and equal depth at the Kingsley Dam in Nebraska. However, where driving is extremely hard, heavier deep channel sections are necessary. Where boulders are present in the soil, it is sometimes extremely difficult to drive even the heaviest sections without curling the ends of the piling. Wood sheet piling can be used only for shallow depths, under the most favorable conditions, and by experienced men. (See also Sections 23 and 24, Chapter 20.)

(*m*) *Drains for Dams on Earth.* Drains serve to carry away harmlessly any seepage that passes under the dam. Layers of rock, broken stone, or holes through aprons are inadequate without the protection of a filter to prevent piping of foundation material through them.

Downstream aprons of concrete are a necessary adjunct for spillway dams on earth. If the main drain shown in Fig. 8 were moved farther downstream, the upstream apron could be shortened, without changing the length of path of percolation. However, this procedure would increase the amount of uplift on the dam and apron, resulting in the necessity for more concrete. Thus the location of the main drain is a matter of economy.

Model tests will indicate the effect of changing the location and depth of the main drain. The deeper it is, the more effective. If there is a relatively impervious stratum at the surface, the main drain should pierce it. For this purpose, deep sand-filled drainage wells were used at the Svirstroy Dam (Fig. 9), since the foundation was composed of horizontal layers of clay with interbedded sandy seams of greater permeability and the whole was subject to severe artesian pressure.

At the Cross Cut Dam (Fig. 11) a water-bearing sand stratum was encountered under the compact gravel, about 10 ft below the base of the dam. To relieve any possible uplift pressure in this area, a filter drain, as shown in the figure, was installed, and 12 well points 14 ft long were added at other places.

Filters under drains must be designed and placed carefully in order to be effective. They consist of layers of materials, each with a porosity less than the layer above it in order to prevent movement of material from one layer to another. Frequently two or more layers are required to obtain the necessary gradation. (See also Section 20, Chapter 20.)

Drains under the apron may not be needed if the main drain is sufficient to eliminate or effectively reduce uplift under the apron and to remove

danger of piping. They were not used at the Imperial Dam (Fig. 2). Possibly the thickness of apron required to withstand impact of the falling water was sufficient to balance any residual uplift.

(*n*) *Downstream Apron*. The purpose of the downstream apron has been explained previously, as well as the necessity for making it heavy enough to balance all uplift pressures. The shape and details of its top surface are frequently governed by the necessity for killing the velocity of the spilling water, as explained in Section 5.

(*o*) *Downstream Cutoff*. The downstream cutoff, as shown in Figs. 6 and 8, is used to protect the foundation under the apron in the event of retro-

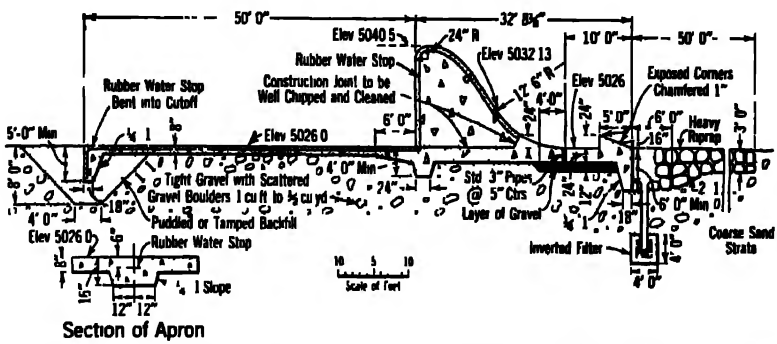

FIG. 11. Cross Cut Dam, Idaho. (J R Sutherland, *Reclamation Era*, July 1938, p. 131.)

gression of the stream bed through failure of the riprap. As explained before, it also reduces the danger of piping where drainage is not controlled.

5. Erosion below Spillways.* (*a*) *Causes of Erosion*. Unless the depth of tailwater is very great and the quality of the rock is especially good, the high velocity of the water at the toe of spillways presents a possible cause for serious erosion. This erosion can occur not only where the steam bed is composed of soft materials, but also, even for relatively low dams, where the bed is solid rock. The velocity is much higher than under natural conditions, and the water is likely to penetrate open cracks in the rock, causing pressures within the seams of the rock that may be equal to or greater than the velocity head of the flowing water.

(*b*) *Stilling Basin for Direct Hydraulic Jump*. Among the methods of dissipating the energy of the falling water, the direct hydraulic jump, explained in Section 14 of Chapter 8, is the simplest and most effective. In that section it was explained that, if the depth of tailwater is less than that required for a jump, the jump will occur some distance below the dam, as in

* Abridged from H. A. Thomas's article "Control of Erosion below Spillways," p. 73, Ref. 1, with data on stilling pools added from Fred W. Blaisdell's article "Development and Hydraulic Design, St. Anthony Falls Stilling Basin," *Proc. A.S.C.E.*, February 1947.

Fig. 12*a*, and the foundation below any provided protection will be subjected to the high velocities.

For such cases, a "stilling basin" can be created by excavating the bed of the river, as in Fig. 12*b*, to provide a depth of tailwater that will produce a

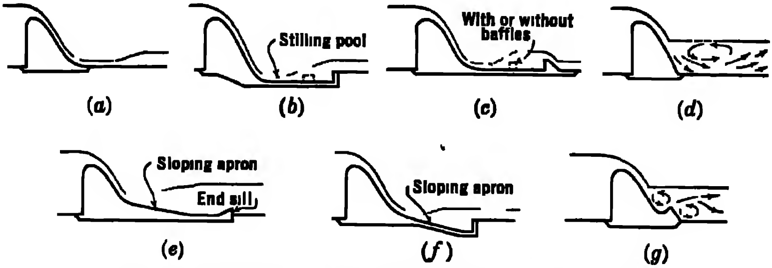

FIG. 12. Spillways with provisions for erosion control.

jump immediately below the dam on the paved area of the bottom of the basin. The length of the stilling basin should be about five times the depth of the tailwater within the basin (Section 14, Chapter 8), unless special provisions, indicated subsequently, are made.

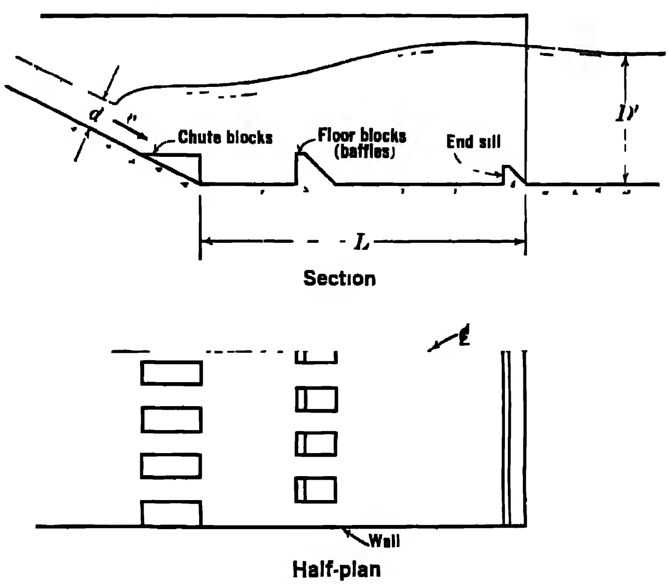

FIG. 13. Diagrammatic sketch of stilling pool.

Although much less used, another method of creating a stilling basin to provide sufficient depth to produce the hydraulic jump, is to build an auxiliary low dam to back tailwater up to the required depth, as in Fig. 12*c*.

Should the depth of tailwater be greater than that required for the hydraulic jump on the river bed, the jump will move upstream and ascend the sloping face of the dam until it is completely submerged by tailwater, as in Fig. 12*d*. Under these circumstances the rate of energy dissipation is slow, as the jet pierces the tailwater and high velocities extend for considerable distances downstream. This condition is frequently remedied by providing a sloping apron with an inclination not exceeding 1 on 4, as shown in Figs. 12*e*, 12*f*, and 14, on which an energy-dissipating jump will occur with a more uniform distribution of velocities downstream.

(*c*) *Stilling Basin for Modified Hydraulic Jump.* If the stilling basin is provided with chute blocks and floor blocks (baffles), as indicated in Fig. 13, a modified hydraulic jump can be created which requires less depth of tailwater than the unmodified case, and the required length of the stilling basin will be less.

According to Blaisdell * if the design and arrangement of chute blocks, floor blocks, and the end sill are properly made, the required general dimensions of the stilling basin are as follows:

Let D = the theoretical required depth of tailwater (high stage);
D' = the required depth of tailwater for modified jump;
d = the depth of the low stage;
v = the velocity at the low stage;
L = the length of the basin;
g = the acceleration of gravity;
$\mathbf{F}$ = the Froude number.

$$\mathbf{F} = \frac{v^2}{gd} \tag{2}$$

$$D = \frac{d}{2}\left(\sqrt{8\mathbf{F} + 1} - 1\right) \tag{3}$$

$$\text{For } \mathbf{F} = 3 \text{ to } 30: \quad D' = \left(1.10 - \frac{\mathbf{F}}{120}\right) D \tag{4a}$$

$$\text{For } \mathbf{F} = 30 \text{ to } 120: \quad D' = 0.85D \tag{4b}$$

$$\text{For } \mathbf{F} = 120 \text{ to } 300: \quad D' = \left(1.00 - \frac{\mathbf{F}}{800}\right) D \tag{4c}$$

$$\text{For } \mathbf{F} = 3 \text{ to } 300: \quad L = \frac{4.5D}{\mathbf{F}^{0.38}} \tag{5}$$

These dimensions are based on the results of 274 experiments on chutes approaching the stilling basins at slopes varying from about 1 on 2.6 to 1 on 1.5 and at velocities ranging from 2.8 to 43.7 ft per second.

* Fred W. Blaisdell, "Development and Hydraulic Design, St. Anthony Falls Stilling Basin," *Proc. A.S.C.E.*, February 1947.

Equations 4*a* through 5 and Fig. 13 apply to spillways less than about 80 ft high. For spillways over 80 ft high, studies should be made of the cavitation potentialities of the design. In the case of high spillways, baffle piers designed without reference to cavitation may give unsatisfactory results owing to disintegration by pitting.

Whether or not a stilling pool is used, an end sill should be provided to deflect the bottom filaments of water from the stream bed.

(*d*) *Sloping Aprons.* As previously pointed out in Section 5*b*, a submerged jump is objectionable. To prevent such submergence, a long sloping spillway apron can be provided, as indicated in Figs. 12*e* and 14.

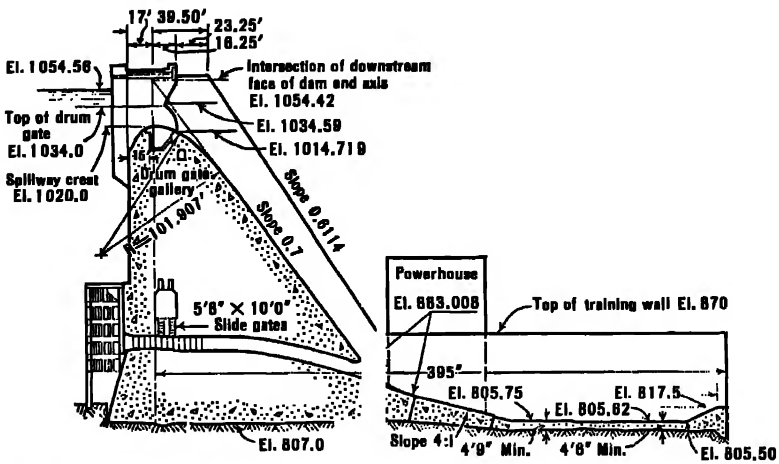

FIG. 14. Section through Norris Dam. (Barton M. Jones, "Design of Norris Dam and Powerhouse," *Civil Eng.*, April 1935, p. 211.)

Experiments on models prove that a hydraulic jump showing strong turbulence in its surface roller and correspondingly excellent energy-dissipating characteristics will form in channels sloping downstream with an inclination not steeper than 1 vertical to 4 horizontal. On slopes steeper than this, the jump tends to take on the submerged form, with milder turbulence in the overlying water, correspondingly slow dissipation of energy from the main jet, and greater bottom velocities.

Where the tailwater depth is greater than that required to produce a jump, the sloping apron must obviously be high and involve a large yardage of concrete, although a portion of this may be regarded as contributing to the stability of the dam.

(*e*) *Upturned Bucket.* Where tailwater depths are more than adequate to produce a hydraulic jump, extremely satisfactory stilling below a high dam can be obtained by means of a strongly upturned and deeply submerged bucket. The one used in the Grand Coulee Dam is shown in Figs. 12*g* and

15. This device involves far less yardage of concrete than does the sloping apron referred to in Section 5*d*. This design should not be used without being subjected to an exhaustive model study, as it involves a feature that might be extremely dangerous. If the tailwater gets too low the jet can push away the tailwater and can rise in the air nearly to the elevation of the headwater. Such action could destroy a powerhouse or other property in the vicinity. Even if the tailwater is adequate for safe operation during floods, the designer should consider the possibility of large quantities of water being thrown high in the air owing to the failure or accidental opening of one or more of the head gates.

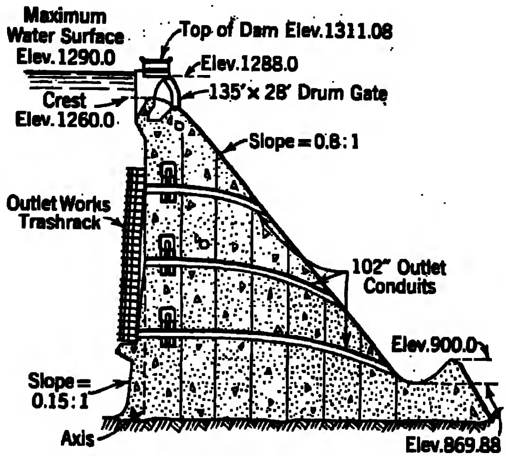

FIG. 15. Spillway of the Grand Coulee Dam.

(*f*) *Arch-dam Spillways*. Arch dams are frequently designed with a vertical, or nearly vertical, downstream face. If the spillway is on the crest of such a dam, the jet leaves the face of the structure and falls freely in the open air. If an artificial stilling pool is not provided, the jet will probably excavate a deep pool of its own, resembling the pool at the base of a natural waterfall. For complete stilling of a vertically falling jet, a great depth of water is required. The stilling pool at the Calderwood arch dam was

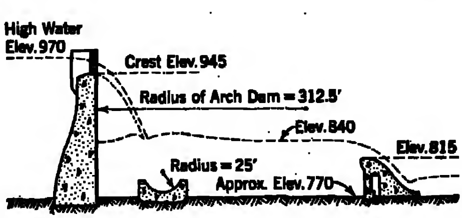

FIG. 16. Stilling basin of the Calderwood arch dam.

created by the construction of an auxiliary dam a short distance downstream from the main structure. As shown in Fig. 16, a curved concrete bucket was installed at the bottom of the pool in line with the trajectory of the jet, to receive the impact of those filaments penetrating to the bottom and to turn them up into the overlying water. The bucket is wide enough to accommodate the trajectory of the jet at all discharges. Model experiments indicated that the jet lost little of its energy before striking the bucket, but the subsequent energy destruction was complete. This stilling system at the Calderwood dam has functioned successfully during several severe floods.

(*g*) *Low Dams*. It was shown in Section 14 of Chapter 8 that, when $(v^2/2g) < d$, i.e., $D < 2d$, i.e., $F < 3$, an "undular" jump occurs, even though D conforms to Eq. 3, and relatively higher velocities persist farther downstream than in a "direct" jump. This condition is likely to occur in very low dams where v is small and d is large. The only remedy is the use of floor blocks (baffles) and other appurtenances to dissipate the energy, as previously described, and a long apron supplemented by downstream riprap as shown in Fig. 2. On account of the great thickness of the jet in such cases, baffles are sometimes omitted because they would have to be quite high to be effective. Fortunately, in dams on permeable materials the requirement of a flat percolation gradient makes the total base width of the structure—dam and apron—sufficiently great to supply space for energy dissipation over the apron.

(*h*) *Retrogression*. Preceding sections have described the necessary provisions for destroying the energy of the water spilling over the dam and for reducing tailwater velocity to the closest possible agreement with the velocity under natural conditions. However, it is obviously impossible to reduce the velocities leaving the protective works to less than the natural velocity for a given flow, and natural velocities frequently scour the bed of the stream. That is, under natural conditions, the beds of many streams lower during the rising stage of a flood and build up again during the falling stage by deposition of material carried by the flood. Owing to the presence of the reservoir created by the dam, all this material carried by the stream deposits in the deep water above the dam and is not available to return the stream bed to normal elevation after a flood, and permanent retrogression results.

For these reasons, the bed of the stream adjacent to the downstream end of the protective works must be treated so that, when retrogression occurs, the foundation for the lower end of the works will not be affected. A usual procedure is to provide a cutoff of concrete or piling at the toe of the protective works, supplemented by riprap of large stones (see Fig. 2). The riprap below the apron is intended to settle and pave a new slope from the toe of the apron to the new level of the stream, and the sheet piling serves to protect the apron from undermining in the event that the riprap directly at the toe settles somewhat.

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CHAPTER 26

SPILLWAYS AND HEADWATER CONTROL

1. Spillways. All power dams are provided with spillways to discharge the excess flow not used by the turbines or stored in the reservoir. Spillways may be divided into the following types:

1. Spillway dams.
2. Side-channel spillways.
3. Chute spillways.
4. Saddle spillways.
5. Shaft spillways.
6. Siphon spillways.
7. Emergency spillways.

All these types except the siphon spillway may consist of an unobstructed overflow, the water rising and spilling over the crest as the flow increases. Very frequently, however, the elevation to which headwater may be allowed to rise is limited by land or water rights which are not owned. Therefore, with such a simple overflow spillway, the crest of the dam must be limited in elevation to provide sufficient margin for the rise of water surface during floods. This, of course, requires a sacrifice of head which otherwise could be used for the generation of power. For this reason, the simple spillway should be as long as possible, consistent with economy, to limit the head on the crest during floods. If sufficient length of simple crest cannot be obtained, these types of spillways may be equipped with crest control or crest gates that can be made to control and limit the fluctuations of the elevation of headwater surface within narrow margins.

The necessity for confining the fluctuations of headwater within very narrow limits exists only when the land for flowage of water rights is absolutely limited or very expensive and the head for power is sufficiently valuable to permit the expenditure for efficient headwater control devices. Such devices are very expensive for control within narrow limits if the flood discharges are great. Obviously, the need for close control is greater in low- than in high-head plants because, for high-head developments a few feet of additional head increase the output a relatively small amount. Therefore, devices that only partly control the fluctuations are frequently used, since they are relatively less expensive. In addition, sluice gates may also be used.

The type of spillway to be chosen in any particular case is usually determined by topographical, geological, and hydrological conditions at the site. For some sites it is clearly evident that only one type of spillway is applicable,

but usually it is desirable, in preliminary studies, as soon as subsurface data are available, to make rough designs for the several types of spillways which might fit the situation and then to select the one that seems to be the most economical, all factors considered. This is not as simple as it seems. It is usually not difficult to estimate with reasonable precision the probable cost of a solid gravity masonry overflow dam section in the center of the valley, but it is quite easy to be misled by the apparent saving of other types of spillways, such as side-channel and siphon spillways.

Maintenance must also be considered in making a decision. For example, a saddle spillway remote from the main dam may appear to be extremely cheap and desirable. However, if rock is eroded from the spillway channel and dumped into the tailrace whenever the spillway goes into action, thus raising the tailwater level and decreasing the head, this scheme does not appear quite so attractive (see Section 4). Therefore the spillway scheme which appears cheap may not be economical in the long run.

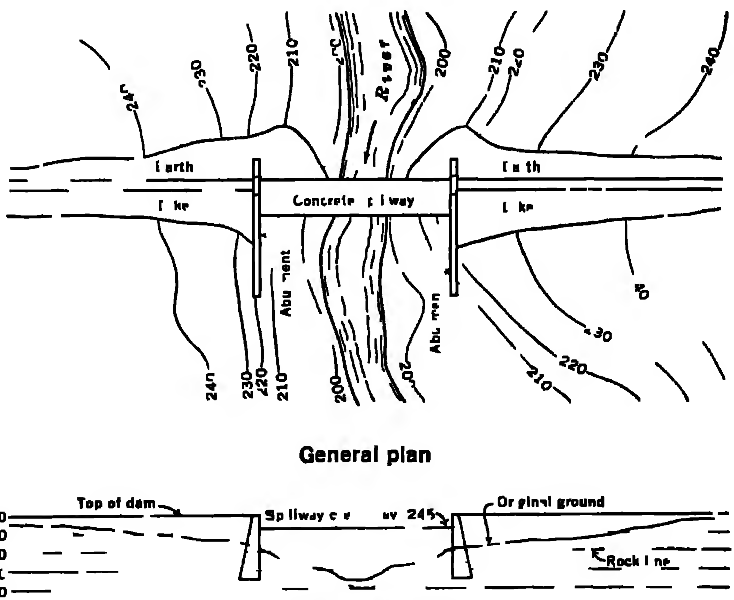

Fig. 1. Diagrammatic example of earth dam with concrete spillway and abutments.

2. Spillway Dams. The design of solid gravity spillway dams and the shape of the crest are covered in Chapter 17, Sections 32 to 40 inclusive; buttressed concrete dams which may be used as spillways are discussed in Chapter 19; flow over dams, in Chapter 8, Section 11.

It is often desirable and economical to use a portion of the dam as a spillway. In such cases it is usually desirable to deliver the spilling water to the stream bed at a velocity equal to or not much greater than that which the stream had before the dam was built, so that erosion is limited. This may be accomplished by means of various devices, such as stilling basins or upturned buckets (see Chapter 25, Section 5).

If the main dam is of earth or rock fill, the situation is quite different. For such a dam, say 50 to 60 ft high, it is generally not too expensive to

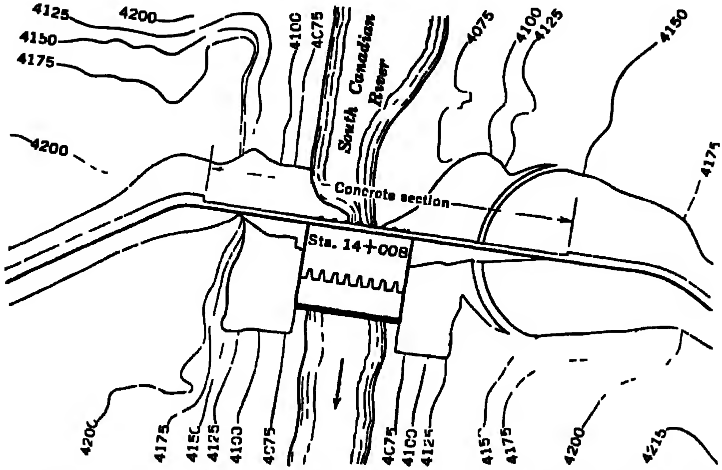

Fig. 2. Earth dam with concrete spillway and non-overflow sections (Conchas Dam, U. S. Engineer Dept.)

have a concrete overflow section in the center of the valley flanked by abutment walls which retain the earth or rock-fill dam on each side. Such an arrangement is shown in Fig. 1.

For a high earth dam, say one of 100 to 300 ft, the size and extent of the retaining walls would be so great as to make the above treatment impracticable. If a remote location for one of the types of spillways discussed in Sections 3, 4, and 5 is at a premium, an overflow spillway may be placed in the center of the valley with non-overflow sections on each side of the spillway which extend just far enough to permit the earth dam to curve around and slope down along the non-overflow sections. Such an arrangement is shown in Fig. 2.

3. Side-channel Spillways. In general, a side-channel spillway consists of a spillway channel parallel to the crest of the weir over which the water enters the channel. A typical side-channel spillway is indicated in Fig. 3. Side-channel spillways are applicable to narrow gorges and sometimes to glacial-filled valleys where only one wall of the valley is ledge

rock. Information on the design of side-channel spillways is given in Refs. 1 and 2 of Section 16.

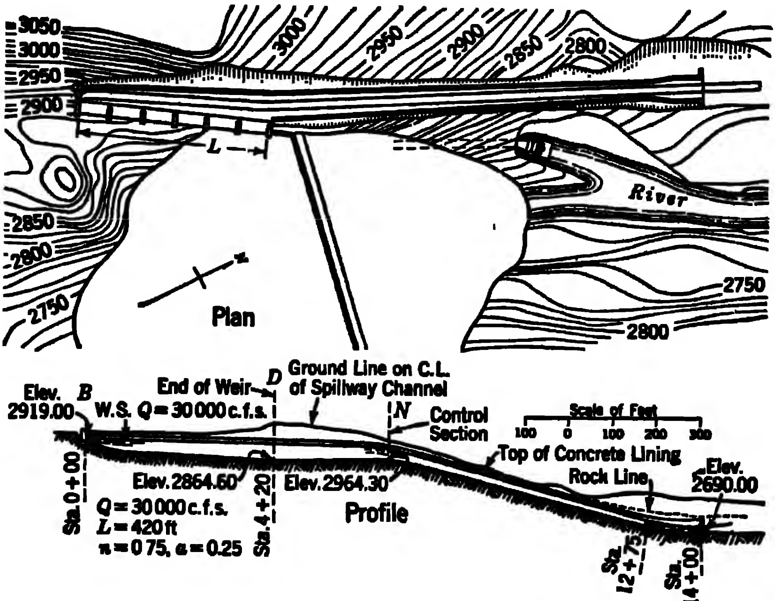

FIG. 3. General plan and profile of side-channel spillway. (Julian Hinds, *Trans. A.S.C.E.*, Vol. 89, p. 882, 1926.)

4. Saddle Spillways. In some basins formed by a dam there may be one or more natural depressions or saddles in the rim of the basin, more or less remote from the main stream channel, where the topography dips to approximately the elevation of the desired flow line. Figure 4 illustrates such a typical saddle-spillway location. To be useful as a saddle spillway it is usually necessary for the saddle to be firm ledge rock.

The possession of a site which would permit the use of a saddle spillway is desirable as it may effect a material saving in cost. For instance, in some cases it may permit the selection of a relatively cheap earth or rock-fill dam as the main dam across the stream valley and the use of such a saddle for a spillway. In others the cost of such a saddle spillway is entirely negligible since nature has already cut the spillway channel. Mohawk and Tappan Dams in Ohio, Bear Creek Dam in Pennsylvania, Conchas Dam in New Mexico, Saluda Dam in South Carolina, and Narrows Dam in North Carolina are typical examples of dam and reservoir sites with saddles.

For a flood control dam where the spillway may be overtopped only once in many years, a saddle spillway is worth while and may result in a material saving in cost. Considerable erosion will take place below a saddle spillway

if, as is usually true, it is not protected from scour. If this erosion will deposit in the river so as to reduce the head on the power development, or will be otherwise objectionable, a saddle spillway would not be justified. In many developments where a saddle spillway is used, the tailrace must be excavated every one or two years. In others, head is permanently lost because deposits are so great that it would not pay to remove them completely.

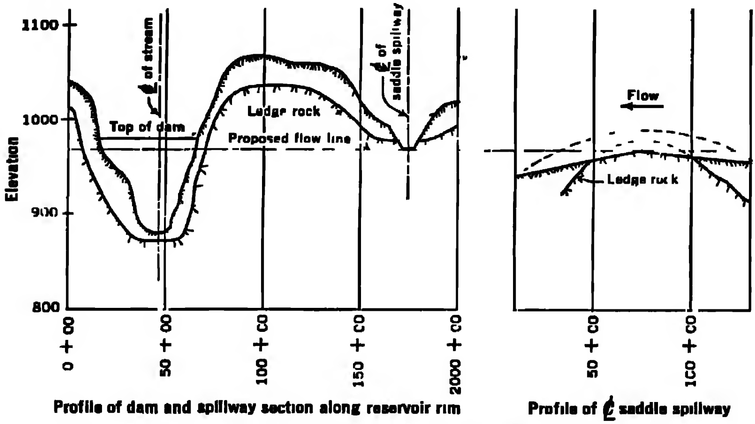

FIG. 4. Typical saddle spillway location.

5. Chute Spillways. The term "chute spillway," as here used, refers to a spillway isolated from the dam, having its crest normal to its center line, as indicated in Fig. 5, and having a discharge channel to the river in an excavated trench which is usually paved with concrete in whole or in part. This type of spillway is generally adopted where it is impossible or undesirable, for special reasons, to pass floods over the dam, particularly a dam composed of earth or rock. Chute spillways are generally located where the contours, both earth and rock, are best suited for economical construction.

The spillway crest may be straight, as in Fig. 5, but an arc normal to flaring side walls may be found more desirable. Where the crest is not a flat-crested weir but is a weir section of some height, the curved crest offers better hydraulic features, since it prevents disturbances downstream as the discharge over the crest is not parallel to the side walls.

The required length of crest is subject to economic consideration since, for a given elevation of crest, the height of the dam increases as the crest is shortened. The spillway is usually widest at the crest and then narrows to a width which is determined by the most economical shape of the discharge trough. The extreme lower end is sometimes flared or widened to reduce the energy per linear foot of the water entering a stilling basin.

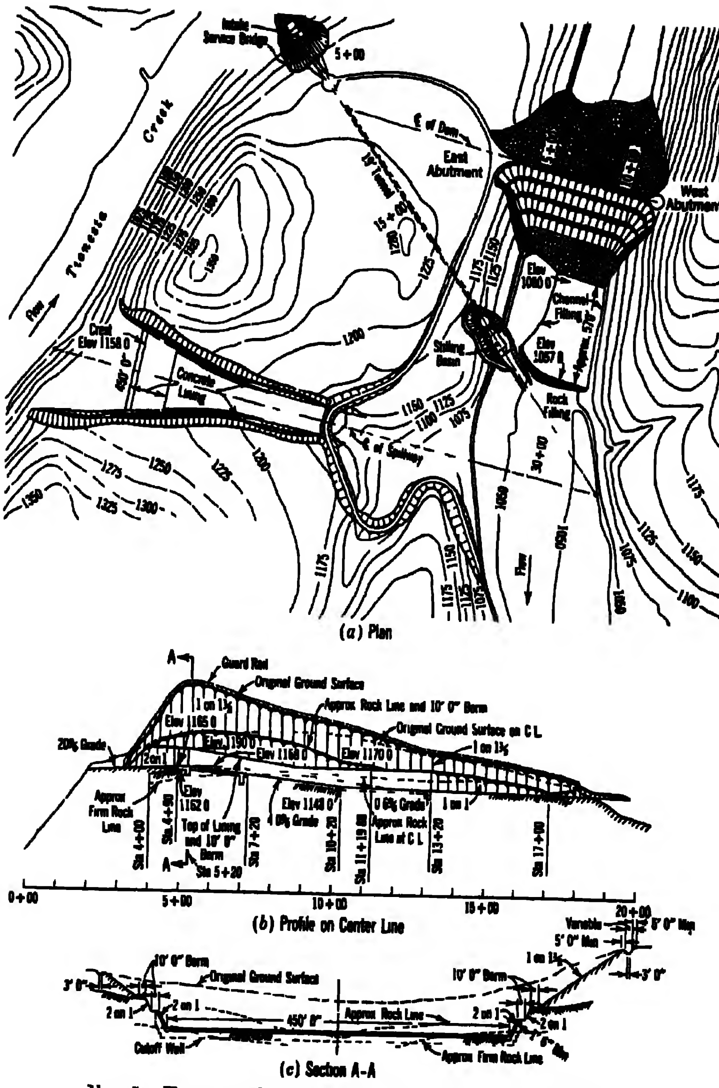

Fig. 5. Tionesta chute spillway. (U. S Engineer Dept)

Varied flow characteristics of chute spillways may be determined by methods given in Chapter 8, Section 6. For details of design, see Refs. 1 and 3, Section 16.

6. Shaft Spillways. A shaft spillway, sometimes termed a "morning-glory spillway," consists of a vertical, flaring funnel, as shown in Fig. 6, with

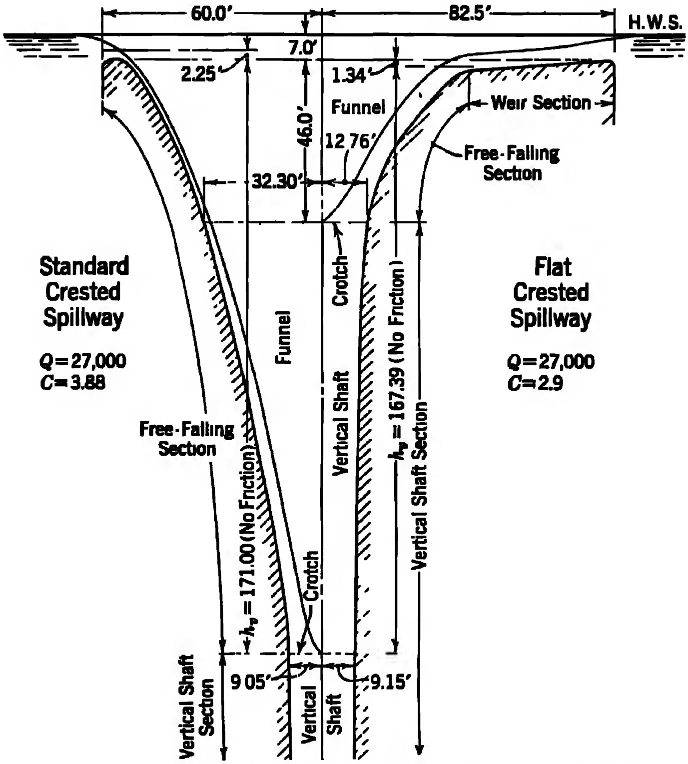

FIG. 6. Comparison of standard- with flat-crested shaft spillway.

its top as the lip of the spillway. The funnel connects with an ell-shaped outlet conduit extending through or around the dam.

There are two general types of shaft spillways, the first having a standard crest (see Chapter 8, Section 11) and the second a flat crest. These are compared in Fig. 6. The flat-crested spillway consists of a "weir section," a "free-falling section" where the shape of the spillway conforms to the path of a free-falling jet, and a "vertical-shaft section," which is completely filled with water. Below the vertical-shaft section are the elbow and horizontal conduit.

In the standard-crested spillway the outline is the same except that there is no weir section, the water beginning its free fall immediately upon leaving the crest, whereas in the flat-crested spillway the water is caused to approach the crest on a flat slope before beginning its free fall.

The standard-crested spillway has the advantage of a smaller-diameter crest, since its coefficient of discharge is greater than that of a flat crest, the former diameter being only about 75% of the latter. Other requirements being met, the standard-crested spillway is therefore more advantageous if the spillway is a tower.

However, the flat-crested spillway has the smaller funnel, the diameter of that shown in Fig. 6 at a distance of 46 ft below the crest being only about 39% of that required for the other type. Therefore, the flat crest is always preferable where the spillway is excavated in the rock.

The vertical-shaft section of the standard-crested spillway is always somewhat smaller in diameter since the head available for vertical velocity, h_v, is somewhat greater. However, this difference is usually negligible.

The height of the free-falling section (Fig. 6) may be greater than the available vertical distance to the elbow. When that is true, even though the spillway is a tower, a flat-crested or a composite spillway may have to be adopted, or the head on the spillway increased, resulting in a smaller diameter, and the reservoir level maintained by gates on the crest.

For details of design of shaft spillways the reader is referred to Refs. 1, 4, and 5, Section 16. Possible accuracy with present methods of design is not sufficient to proceed without the assistance of model tests. It is also necessary to determine by such tests the location and amount of negative pressures which may obtain not only for the design head but also for lower heads.

It is suggested that, for a standard-crested spillway, a preliminary test be made with a circular sharp-crested weir, equipped with piers if required. The shape of the jet having been determined, the model can be completed and the final test made.

For a flat-crested spillway, the preliminary test would be conducted by installing only the weir section (Fig. 6) and measuring the shape of the under-nappe.

7. Siphon Spillways. This type of spillway is a device for discharging water through a closed conduit based on the principle of the siphon. It utilizes the available head at the dam to produce a higher velocity of flow than would be attained at an overflow weir, thus increasing the discharge for the same elevation of water surface in the reservoir. Thus, for a given length available for a spillway, the use of a siphon spillway will result in a smaller rise of water surface for the same discharge than would be possible with an overflow weir.

Figure 7 shows the normal headwater surface at the level of the crest of the siphon spillway. When the water rises, it spills over the crest; and, when the flow is such that the discharge strikes the downstream side of the lower

leg, the air thus confined in the throat is quickly entrained and ejected, and the siphon primes. The suction thus produced increases the velocity to that corresponding to an effective head equal to the difference in elevation between headwater surface and the center line of the outlet, less the head expended in friction within the siphon.

When the upper parts of the air vents are exposed, the air drawn in by the suction reduces the efficiency of the siphons until the discharge is automatically diminished to that required for stationary water surface in the pond. If, however, too much air is drawn in, the siphon action will be broken and the headwater surface will then rise again, and the operation of priming will be repeated. If properly proportioned, the siphons will prime within a few seconds after the water has risen to the required elevation.

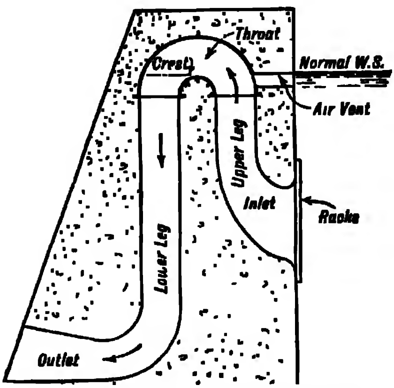

FIG. 7. Simple siphon spillway.

The upper leg is made of sufficient length to bring the inlet well below water surface, in order to prevent the entrance of ice and drift. The inlet is made two or three times the area of the throat, is well rounded, and is usually protected by rack bars rather wide apart. The lower leg should be as long as is practicable, up to the siphonic limit, to take advantage of all the head available. The outlet may be submerged or may be opened to the air, except for special cases where submergence is necessary.

Siphons with a throat as great as 8 ft have been used. The throat is frequently protected by a lining to prevent cavitation. Generally, siphons are designed to prime before the water rises too high, usually a distance equal to about one third the height of the throat. However, it is more practical to design a siphon spillway so that it will prime for a headwater differential of 5 or 6 in. This can be done by sealing the outlet and providing a small auxiliary siphon to prime with a 5- or 6-in. rise and produce a suction to draw the air out of the 8-ft siphon.

Some very large siphons have been designed with capacities as great as 18,000 sec-ft, and with a head between headwater and tailwater equal to 175 ft. The design of such siphons is rather tricky. The designer must be sure to get the lower leg of the siphon of small enough diameter to create sufficient resistance and prevent a vacuum from breaking the column of water in that leg. Probably long before this point is reached, the excessive vibration would be enormous. If anything but a simple and small siphon is contemplated, model tests should be a prerequisite of design. In general, it is believed other means of discharging the desired quantity of water past a dam would usually be found more economical and desirable.

The discharge of a siphon may be computed from the ordinary equation for flow in short tubes (Chapter 8, Section 1, Eq 2). Here a is the area of the throat, and the value of the coefficient C ranges, for extreme cases, from 0.25 to 0.98. Siphons similar to that shown in Fig. 7 have a coefficient of about 0.65. Methods to determine the coefficient C are identical with the theory of the flow of water in pipes and are described in Ref. 6, Section 16.

8. Emergency Spillways. An emergency spillway is one that will be called upon to operate so infrequently that it is not considered necessary to protect the spillway control, the structure, its foundation, or its discharge

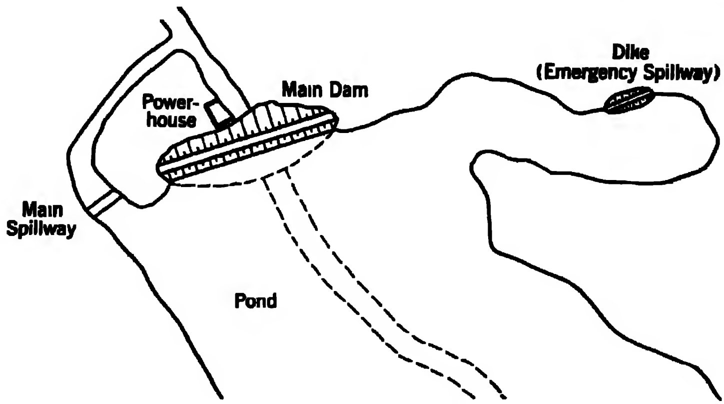

FIG. 8. Typical emergency spillway arrangement.

channel from serious damage when it goes into action. However, further definitions of this type of spillway vary according to two points of view.

From the first viewpoint, the emergency spillway is an auxiliary spillway which would be called into action should a flood greater than the spillway-design flood occur. From this viewpoint it is simply an added factor of safety.

From the second point of view, the emergency spillway is an auxiliary spillway which would be called into action should a flood occur whose magnitude was, for example, 60 to 80% that of the spillway-design flood. That is, a permanent spillway would be built to accommodate 60 to 80% of the proper flood, and an emergency spillway would be provided to take the remainder with considerable damage to it but with safety to the dam.

An emergency spillway is most easily obtained at a low divide in the reservoir rim. If the elevation of the divide is so low that a dike is required, the dike is left at such an elevation that it will be overtopped before the main dam is overtopped.

A typical emergency spillway is shown in Fig. 8. The elevation of the top of the small dike, located at a low divide, is placed at the maximum elevation of water surface corresponding to safety for the dam, the top of the

dam being, of course, at a higher elevation to accommodate waves and to provide that, should a greater rise of water surface occur, the dike would be washed out first. However, the dike should be of such a nature that it would not erode fast enough to lower the pond too rapidly.

The dike of the emergency spillway shown in Fig. 8 is well protected from wave action. If it is not, the dike must be protected by a breakwater, since the elevation of water surface at which it would fail must be fixed closely.

Should there be no low place in the divide requiring a dike, an emergency spillway can be provided if a channel cut through a divide is not too expensive. In such cases, the channel should be cut to a depth sufficiently below high-water surface to provide the required capacity, and the opening should be closed by a dike built to the proper elevation, as previously described. In very erodible materials, it may be possible merely to excavate a pilot channel and let the flood excavate the rest. In such cases, however, care must be taken to avoid the possibility of a side-slope slide which would block the flow of water and render the spillway ineffective.

9. Headwater Control.* *(a) General.* The devices commonly used for the control of the elevation of headwater, as mentioned in Section 1, may be divided into the following classes:

1. Crest control, which lowers or raises the crest as the river discharge varies.
2. Crest gates, which are opened and closed to vary the capacity of the spillway as desired.
3. Sluice gates or valves placed in passages through the lower part of the dam which, usually in a relatively small measure, augment the spillway capacity and which primarily serve to pass needed water through the dam when insufficient water is being used by the turbines or being spilled.

The foregoing classes may be further divided into the following usual forms or types:

1. Crest control.
   1*a*. Temporary flashboards.
   1*b*. Permanent flashboards.
   1*c*. Tilting gates.
   1*d*. Bear-trap crests.
   1*e*. Drum crests.
2. Crest gates.
   2*a*. Plain sliding gates.
   2*b*. Taintor gates.
   2*c*. Wheeled gates.
   2*d*. Stoney gates.
   2*e*. Caterpillar gates.
   2*f*. Rolling gates.
   2*g*. Stoplogs or needles.

* Sections 9 to 13 inclusive are condensed from Chapter 24, W. P. Creager, Joel D. Justin, and Julian Hinds, *Engineering for Dams*, John Wiley & Sons, New York, 1944, by Charles M. Wellons, Technical Assistant to District Manager, U. S. Engineer Office, Pittsburgh, Pa.

3. Sluice gates and valves.
   3a. Plain sliding gates.
   3b. Wheeled gates.
   3c. Caterpillar gates.
   3d. Butterfly valves.
   3e. Needle valves.
   3f. Cylinder gates.
   3g. Radial gates.

(*b*) *Choice of Type.* Not only is it impossible to describe within a limited space all the many devices used for the control of headwater, but also it is very difficult to define all the contingencies that affect the choice of type. Where water is very valuable, it is essential to adopt a type that is tight and will not waste the flow during periods of drought. For rivers containing much ice or a large amount of débris during floods, the crest must be unobstructed or provided with piers that are sufficiently far apart to prevent stoppage.

In this country, flashboards, Fig. 9, are the most common of all headwater control devices for small dams. They have been used up to 10 ft in height, although a maximum height of about 4 or 5 ft is usually installed. The chief objection to both temporary and permanent flashboards is the difficulty of setting them up again after they go into operation.

The tilting type of gate and the bear-trap crest, Fig. 14, have been superseded largely by the drum gate, Fig. 16. The drum gate is useful where a large amount of débris or ice is to be passed by the dam without fully opening the gates. Lift gates require full opening to clear the surface of the water, but drum gates need to be lowered just the amount sufficient for that purpose. The large Shasta drum gates, Fig. 16, are 28 by 110 ft.

Sliding lift gates, Fig. 24 of Chapter 27, are seldom used on account of the large capacity of hoist required to overcome the excessive friction. It is usually found more economical to use fewer piers and adopt a large gate of a type easily lifted.

Taintor gates, Fig. 17, are the most common type of crest gate. They are not quite so tight as the straight lift type of gate but can be made so with a little maintenance. The usual kind has been used for spans up to 64 ft and for heights up to 35 ft. Much larger gates would seem feasible. However, wheeled gates have been used for larger sizes.

Wheeled gates, Figs. 26, 27, and 28 of Chapter 27, have been used up to spans of 80 ft, but, on account of the required wheel spacing, a value of span (in feet) times head on bottom of gate (in feet) of about 2000 is close to the limit of usual practice.

The development of heavy-duty roller bearings and heat-treated wheels and rails has led to the general use of the wheeled gate in preference to the stoney gate, Fig. 30 of Chapter 27, which formerly was adaptable to equal sizes.

The caterpillar gate, Fig. 18, has been used for crest gates up to a span of 30 ft, but is more frequently used for sluice and intake gates

Rolling gates, Fig. 19, have been used to only a limited extent in this country. They are adaptable to spans up to 150 ft and heights up to 30 ft where exceptional amounts of débris or large ice flows require a wide clear opening.

Stoplogs and needles have been used principally for small trash and ice chutes and for temporary closure of openings for maintenance of gates.

10. Crest Control. The devices used for crest control, i.e., to adjust the elevation of the crest as previously explained, make use of the headwater pressure to raise or lower the damming structure and utilize relatively small differences in elevation of headwater to regulate discharge automatically.

(*a*) *Temporary Flashboards.* Flashboards consist of a series of vertical boards or panels placed on the crest of the dam for the purpose of raising the pond level. Figure 9 shows a typical system of temporary flashboards, consisting of a series of panels supported by pins or pipes which are inserted loosely into sockets set in the masonry crest of the dam. The pins or pipes are designed to bend over and loosen the flashboards when the water surface in the pond reaches a certain elevation, thus automatically lowering the crest to pass excessive floods. The boards or panels are fastened loosely to the supports, generally by nails driven through the boards and bent around the pins, and are lost when the supports bend over, unless removed before anticipated high water. If they remain on the crest, the effective elevation of the crest to pass floods will be raised.

In order to facilitate the handling of a barge for removing and restoring the flashboards, it may sometimes be found desirable to provide, at intervals, sockets into which mooring pins may be set, as indicated in Fig. 9. Cableways may also be used as an anchorage for the barge. The boards and supports are sometimes manipulated from an overhead trolley or bridge on the crest of the dam.

Where it is possible to remove the flashboards in advance of floods, they are usually built in panels, as indicated, and provided with handles. For the type shown, handles are not permissible if considerable drift is anticipated, as the drift may collect and cause premature failure. The boards usually have unplaned edges, and ashes or similar calking materials are used to make them tight.

Flashboard pins are expected to carry a certain predetermined load and to fail when that load is exceeded. The elastic limit, ultimate strength, and behavior of commercial solid rolled-steel bars, when the elastic limit is exceeded, are quite variable and the load under which failure will occur cannot be determined with any degree of exactness. A closer limitation of the range of load under which failure of solid pins will occur is obtained experimentally by turning a circumferential notch at a point that will come immediately above the socket. Notching requires special fabrication, which may not always be done in ordinary replacement.

Pins made of commercial steel pipe will bend over completely and very abruptly when overloaded and are therefore preferable to solid pins. Al-

though more reliable than unnotched solid pins, double-extra-strong pipe is not as dependent as extra-strong or standard pipe. In some cases, the pipe pins are also notched in order that they will not bend over but will snap off and pass with the flood, leaving the crest entirely unobstructed. However, this is not considered necessary if the flashboards are removed by the flood.

Tests on different sizes of galvanized-steel and wrought-iron pipe, made by the Bureau of Standards in cooperation with the U. S. Forest Service [8], indicate that the average stress at failure * was between 67,000 and 81,000 lb per sq in. with an average of about 75,000 lb per sq in. The greater strengths were found for the smaller pipe. Just before flashboard pins fail,

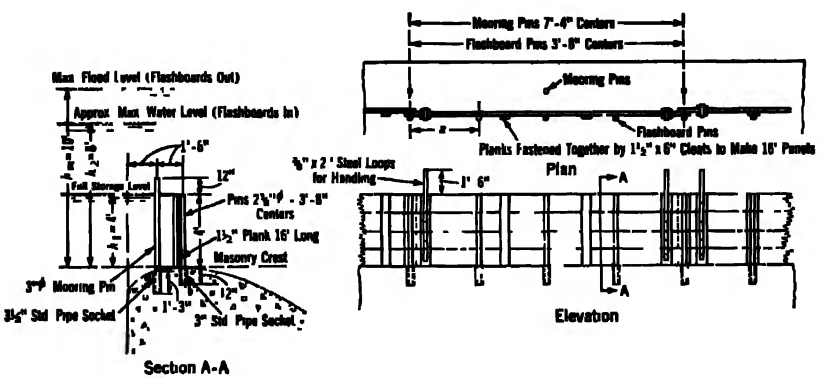

FIG. 9 Temporary flashboards.

while water is passing over the top of the boards, there is a considerable but unknown amount of vacuum under the jet which increases the stress on the pins. Therefore the calculated stress in the pins at failure, neglecting this vacuum, is considerably less than the ultimate strength indicated by tests.

It is recommended that flashboard pins be designed for an initial stress of about 20,000 lb per sq in. when the water surface is at the level of the top of the flashboards, where they are subject to the pressure of waves and débris and ice and therefore liable to premature failure. However, if they are well protected, the use of an initial stress of 30,000 lb per sq in. would be permissible.

Failure will occur when the final stress due to rise of water surface and vacuum under the jet becomes about 75,000 lb per sq in. The amount of vacuum is indeterminate, but tests on actual flashboard failures indicate that a design based on a stress of 50,000 lb per sq in. would approximately take care of the force of the vacuum, if it is neglected in the assumed forces acting.

* Based on straight-line formula.

For an example, let us assume that it is possible to raise the water surface during floods to an elevation 100.0, and that a higher elevation of water surface would flood lands not owned. Assume that a 6.5-ft depth is required to pass the maximum flood over the spillway after the flashboards have failed. Then the permanent spillway crest would be placed at elevation 93.5.

With these data, it is desired to design flashboards which will fulfill the following conditions: (1) they must be as high as possible in order to provide the highest elevation of normal water surface; (2) they must fail before the water surface rises above elevation 100.0, or 6.5 ft above the permanent dam crest; (3) they must not fail so frequently under medium floods as to be uneconomical; (4) they must be reasonably safe from premature failure under impact of waves and débris.

Assuming that the flashboards are fairly well protected from the forces of waves and débris, an initial stress of 25,000 lb per sq in. is permissible.

Adopt tentatively a 3.9 ft height of flashboards, leaving a margin above them of 6.5 minus 3.9, or 2.6 ft. In Fig. 10, at the left-hand margin, find $H = 3.9$ ft. Trace horizontally to intersect $h = 0.0$ ft, corresponding to water surface at the crest of the boards, thence vertically to intersect the initial stress K, of 25,000 lb per sq in., thence horizontally to intersect the desired spacing of pins, say $d = 3.5$ ft, thence vertically to the lower margin, and find a required section modulus S of 1.1 and a standard 2½-in. pipe pin.

Reversing this procedure, it is found that, for $S = 1.1$, a spacing of 3.5 ft, and a final stress K of 75,000 lb per sq in., the water surface would be $h = 2.6$ ft above the top of the $H = 3.9$ ft flashboards when failure occurs, or at $H + h = 3.9 + 2.6 = 6.5$ ft, which is desired, and our tentatively adopted height of boards is correct.

The foregoing assumes that there is no vacuum under the jet. It has been previously shown that the effect of a vacuum may reduce the indicated final stress to 50,000 lb per sq in.

Therefore, repeating the procedure for a final stress of 50,000 lb per sq in., it is found that failure may occur at a head, h, on the flashboards of 1.3 ft.

With this design, one is reasonably sure that failure will occur with between 1.3 and 2.6 ft above the flashboards, with the probable value closer to 1.3 ft.

If it is felt that a flow corresponding to 1.3 ft over the boards will occur so frequently as to make the failure of the flashboards a nuisance, or too great an expense, then the only alternative is to reduce the height of the boards and also the initial stress in the pins.

For instance, the use of a 3-ft height of flashboards of the same size but with a 5-ft spacing and with an initial stress of 16,000 lb per sq in. instead of 25,000 would result in their going out with between 2 and 3.5 ft over them. Thus, the elevation 100.0 will not be exceeded in the event that the vacuum load is not created, and the head h at probable failure has been increased from 1.3 to 2.0 ft with greater flood-carrying capacity before flashboard

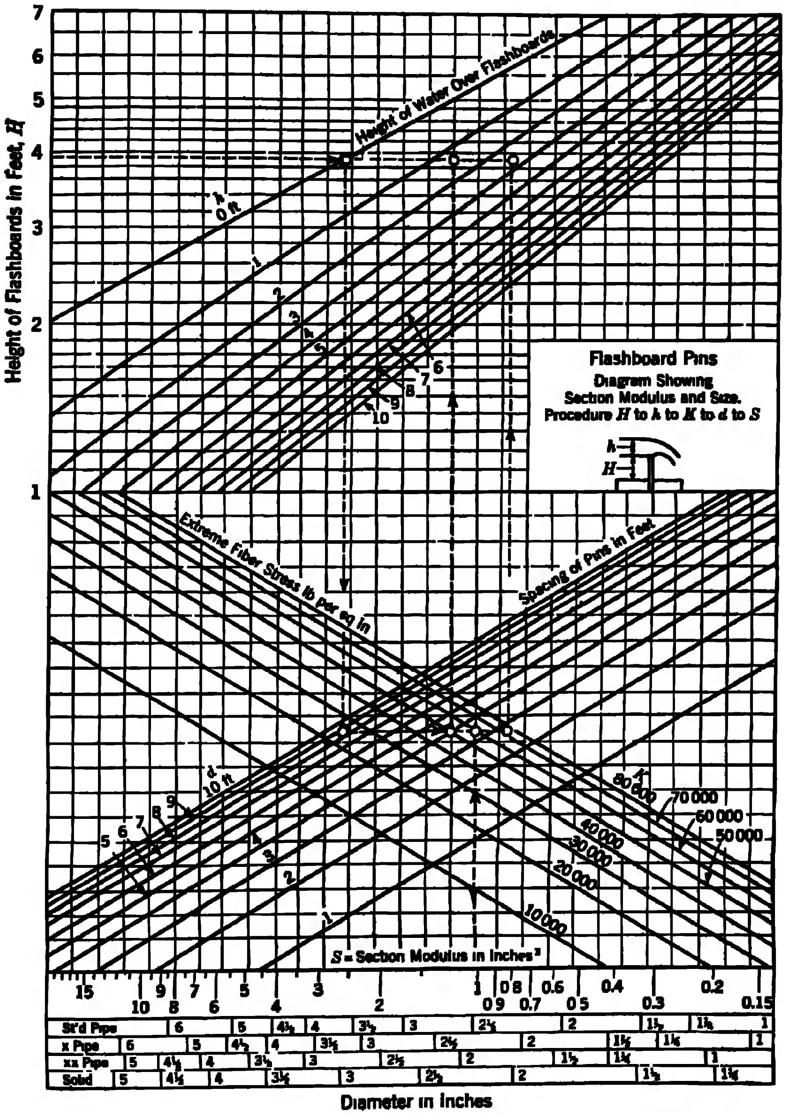

FIG 10 Flashboard pins, diagram showing section modulus and size.

failure, but at the sacrifice of a lowering of the normal water surface 3.9 minus 3.0, or 0.9 ft.

The foregoing theory applies to standard dam crests. Unpublished experiments by Hibbert Hill have shown that, in the case of flat-crested weirs, the deflection of the impinging jet on the horizontal surface at the elevation of the base of the flashboards will set up a dynamic reaction which will raise the water surface under the jet and materially reduce the calculated moment on the pins, as shown in Fig. 11, for values h/H greater than about ⅓.

In some cases, the panels have been arranged as shown in Fig. 12. When the support, A, is removed, the panels revolve progressively around the pins until all the panels are either swept away or left hanging to the pins as shown by the dotted lines. This scheme leaves the pins in place and they can be

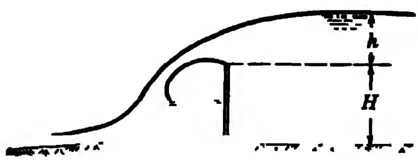

FIG. 11. Effect of vacuum behind flashboards.

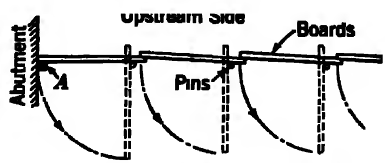

FIG. 12. Plan of flashboards designed for progressive failure.

used over again. However, if the river is subjected to a considerable run of débris during floods, enough of this may accumulate on the pins, after the panels have gone, to obstruct the crest without bending the pins, thereby causing higher water during floods than has been anticipated.

Temporary flashboards are always advantageous for any dam not otherwise equipped with crest control, provided that the pond does not lap the development next above. Pipe sockets should be provided in all crests of such installations, whether flashboards are contemplated in the near future or not.

After the boards are removed by a flood, the operators are usually able to replace them before the flow is reduced to the capacity of the turbines and the pond refilled by the tail end of the flood. With sufficient capacity of sluice gates or other emergency outlets, the water surface can be lowered to the elevation of the permanent crest before the flood has entirely receded, and the flashboards can be easily replaced. The sluice gates are then closed and the pond is filled to the top of the boards.

An ample number of spare pins should be provided since one difficulty in the use of flashboards is the tendency of operators to replace broken, well-designed pins with any piece of handy metal, mostly of excess strength, in order to avoid replacing the pins so often. Such practice is obviously dangerous.

The thrust of ice and, when the pond fluctuates, the lifting force of adhering ice frequently cause premature failure of flashboards. It is therefore

necessary to maintain a channel of open water along the upstream face of the boards. This may be accomplished [10] (1) by cutting the channel by hand or with steam jets or ice saws; (2) by use of a bubbler system consisting of the release of jets of air at a lower elevation to bring warm water to the surface; (3) by use of heaters under covers stretched between the ice sheet and the flashboards.

Ice frequently accumulates on and below the downstream face of the flashboards in an amount sufficient to prevent failure. However, it is considered that, on large watersheds, this ice will be removed by the gradually rising spilling water. On small watersheds where flash floods are possible there may be insufficient time for such removal and the downstream face should be kept free of ice.

(*b*) *Permanent Flashboards.* Permanent flashboards are similar in principle to the temporary type, except that they are designed to operate auto-

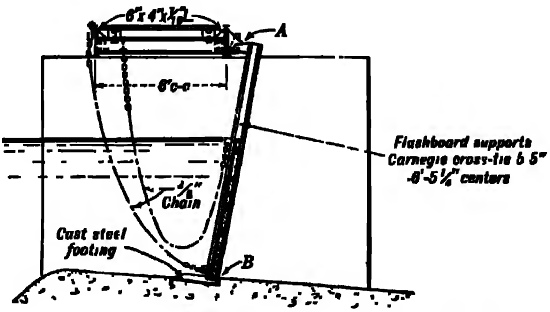

FIG 13. Details of flashboard device, Davis Bridge Spillway. (E. A Dow at Spring Meeting, *A S M E.*, 1925)

matically or by manipulation, without damage to themselves. They have been used mostly for special conditions and have not come into general use.

Figure 13 shows details of the permanent flashboards used at the Davis Bridge Dam of the New England Power Company. The supports are held in place by a seat on the crest at *B* and a latch on the bridge at *A*. They are removed during floods by tripping the latch. The boards, which are in the form of stoplogs, are lost, and the supports are drawn up to the bridge by means of the chains and returned to place after the flood by fastening the top to the latch and swinging them down into the seat at the bottom. The new boards are then forced down from the top.

One reason for using this type of flashboard in preference to the less expensive temporary type previously described is that the Davis Bridge spillway controls an extremely large reservoir. The tail end of the flood would not be of sufficient duration to refill such a large reservoir if the water surface were allowed to drop to an elevation close to that of the permanent crest in order to replace the flashboards, there being no other outlet of large capacity

to be used to discharge the tail end of the flood while the boards are being replaced.

Many other types of permanent flashboards have been used, including hinged flashboards which are braced in an erect position, some device being used to remove the brace when operation is required.

(c) *Tilting Gates.* Tilting gates of many types have been installed in the past, most of which were patented devices. They are seldom if ever used now. They consist of a hinged leaf on the crest, which is counter-balanced and designed to tilt and release the flow as the water rises.

(d) *Bear-trap Crests.* Bear-trap dams were originally developed for sluicing logs through dams used in logging operations on small streams.

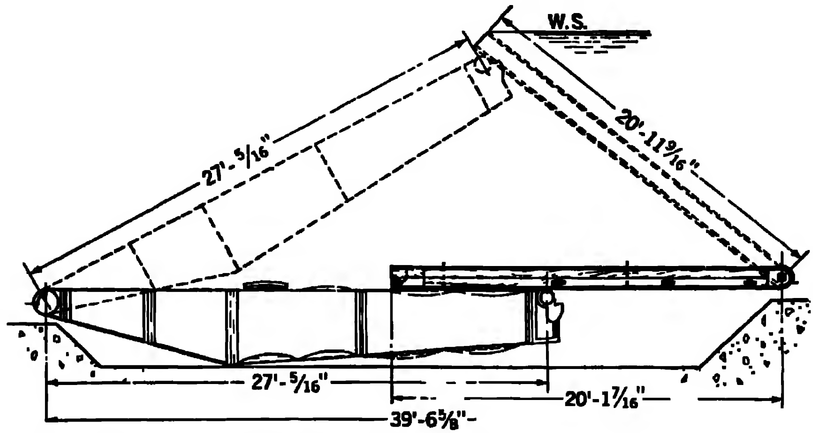

FIG. 14 Bear-trap dam No. 6, Ohio River. (U. S. Engineer Office, Pittsburgh, Pa.)

They have been used for stage regulation on low-head dams, particularly in connection with the movable dams for navigation on the Ohio River.

The Ohio River type of bear trap is shown in Fig. 14. The surfaces and weights of the moving parts are proportioned so that the headwater pressure supports the damming structure. The trap is raised by admitting headwater to the chamber below the leaves and lowered by draining the chamber to tailwater. Bear-trap dams are ordinarily operated in either the fully raised or fully lowered positions. They can be raised against the weight of over-flowing water by the headwater pressure with usual head difference but are arranged to be raised by buoyancy produced by introducing air under the lower leaf when the head difference is small. Bear-trap dams require a wide base but avoid the use of a deep chamber. They are therefore particularly adapted to applications where the fixed crest level is little above stream bed. Bear-trap dams are very difficult to make tight.

(e) *Drum Gates.* Drum gates are hinged gates which, during high water, lower into a compartment in the top of a spillway dam and leave the crest

free to discharge the flood. The simplest form is the Stickney type, shown in Fig. 15, used for the New York State Barge Canal. Its crest is lowered automatically by rising headwater. The leaves are so proportioned that

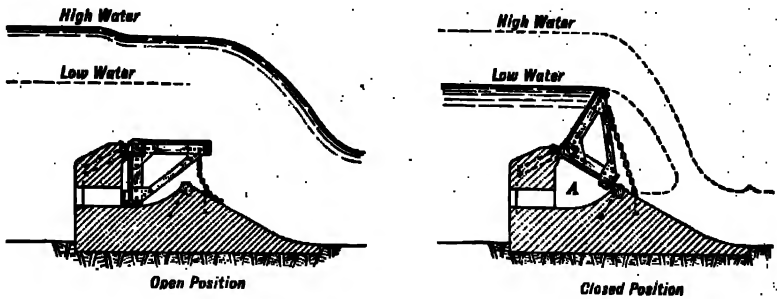

FIG. 15. Stickney type of drum gate.

headwater in chamber *A* supports the movable structure against the headwater pressure above the sill at low water. When the water surface rises,

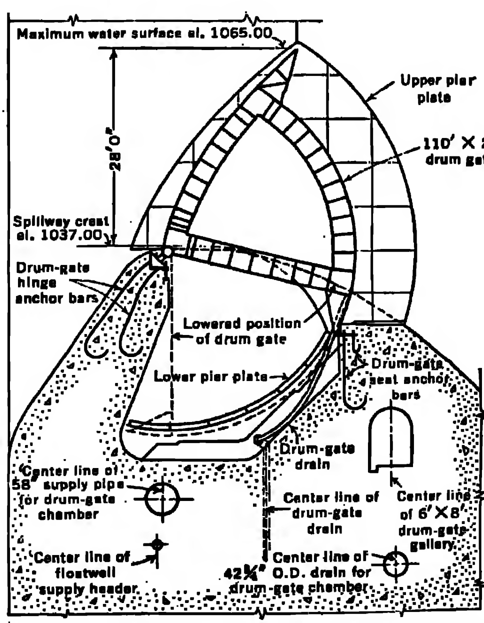

FIG. 16. Drum gate at Shasta Dam. (*Civil Engineering*, September, 1943, p. 428.)

the headwater pressure becomes relatively greater on the upper leaf and the crest is depressed. As the water surface falls to the low water level, the crest returns to that level.

From this modest beginning, drum gates of very large and complicated designs have been made and installed.

Figure 16 shows the type of drum gate used by the U. S. Bureau of Reclamation. It is designed to maintain a controlled crest level and regulated discharge. Control is effected automatically through a device which admits or releases water from the chamber beneath the drum under the influence of the headwater pressure. When the gate is lowered, the top surface of the drum matches the curve of the spillway.

11. Crest Gates. The term "crest gates" is used to designate that class of spillway control in which the damming surface is raised to permit discharge between its lower edge and the fixed crest or sill.

Crest gates are supported between piers placed at regular intervals along the crest of the dam and are operated from a hoist on an overhead bridge. The gates should be capable of being lifted clear of the projecting branches of floating trees, and they are frequently protected by a concrete buffer or front wall above water surface, as shown in Fig. 17. The several types of crest gates usually employed are described in succeeding sections. All such gates of large size are either pivoted or provided with rollers or wheels which reduce the friction enough so that the gates can be opened by cables or chains and closed under their own weight. The friction is so small that counterweights to assist opening will not interfere with closing under the net unbalanced weight.

Crest gates are ordinarily operated by externally applied power and require intelligent control, although they can be, and sometimes are, operated by automatic devices under the influence of the elevation of headwater.

Where only a few lift gates are installed, individual hoists are usually provided; when there are a large number of gates, traveling hoists are commonly used. A spare hoist should always be provided if traveling hoists are used, and several active hoists may be necessary if the gates must be raised quickly during floods. Hoists for crest gates are usually electrically operated, as hand operation for large gates is very slow.

(a) Plain Sliding Gates. Plain sliding gates consist of a flat leaf spanning the opening and resting directly on the gate supports. Gate stems with racks, engaging pinions on the hoists on the bridge, serve to operate them as indicated in Figs. 52 and 53 of Chapter 27. In operation, the friction of the gate on guides, due to the pressure of the water, is large and must be overcome. The gates are frequently made of wood with timber or structural-steel stems. For a typical design of a wooden slide gate, see Fig. 23, Chapter 27. A Bureau of Reclamation all-metal slide gate is shown in Fig. 24, Chapter 27.

(b) Taintor Gates. The taintor gate, as shown in Fig. 17, consists of a wood or, more usually, a steel facing on a framework which is shaped in the form of a sector of a circle. For this reason it is sometimes called a "sector" gate. The whole is arranged to rotate around a horizontal pivot attached to the piers.

Taintor gates are usually raised by means of ropes or chains acting simultaneously at both ends. Since the angular travel in the bearings is small when the gate is raised from the closed to opened position, the work done in overcoming the frictional resistance is small. Counterweighted taintor gates can be made to operate automatically by the addition of fairly simple automatic control devices.

A further description of taintor gates is given in Chapter 27, Section 16.

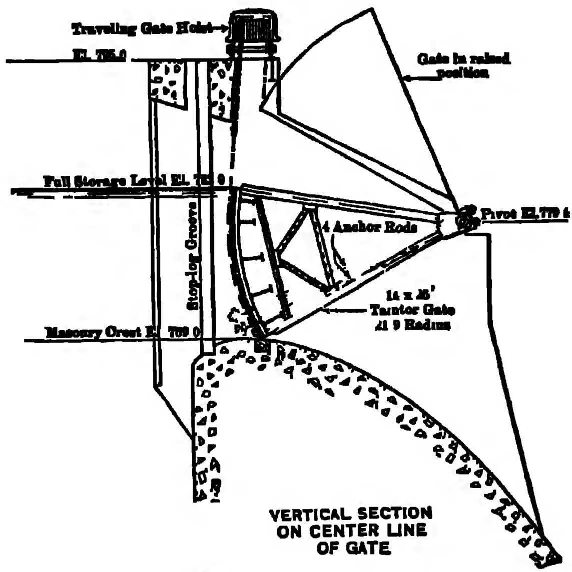

Fig. 17. Taintor gates for Great Falls Power Dam, Caney Fork River, Tenn.

(c) *Wheeled Gates.* Wheeled gates consist of a flat leaf supported by self-aligning roller bearing wheels. They are more fully described in Chapter 27, in which the following figures occur. The wheels may have direct support as in Fig. 30 or cantilever support as in Figs. 26 and 27. An end roller to keep the gate in alignment may be used, as in Fig. 30, or the wheels may run on tracks as shown in Fig. 26.

The roller bearings must be protected from water, air, and abrasive materials. The bearing chambers should be kept packed with grease and are generally sealed by watertight closures which fit snugly about the axles to exclude water and air and to retain the grease.

(d) *Stoney Gates.* Stoney gates bear upon rollers, but the rollers are not attached to the gate. Vertical tracks at each end of the gate ride upon rollers which in turn roll upon fixed tracks in the piers.

The rollers are attached together in "trains" to maintain correct spacing and alinement and to support them in position when the gate is fully raised. The roller train travels only half as far as the gate. Typical stoney gates are shown in Figs. 30 and 31 of Chapter 27.

(*e*) *Caterpillar Gates.* An installation of the caterpillar gate is shown in Fig. 18. These gates are essentially the same as stoney gates except that the roller trains are arranged as continuous chains, as shown in Fig. 29 of Chapter 27.

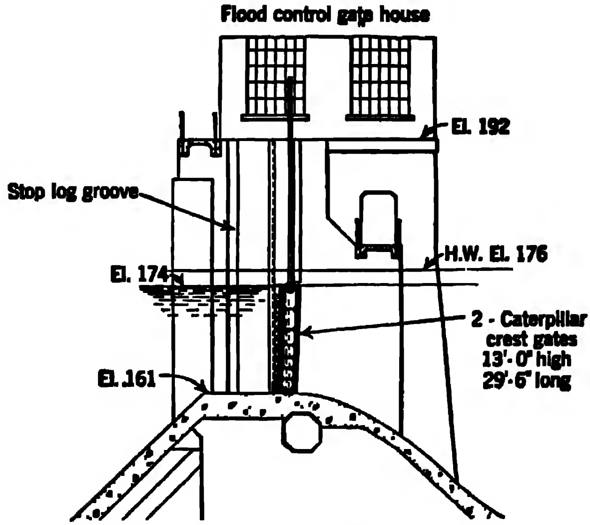

FIG. 18. Caterpillar crest gates, waterworks reservoir project, Oklahoma (Philips & Davies, Inc.)

The rollers are connected by links, and the assembly travels around a continuous track that is framed into the end of the gate. The broome-type gate makes use of this construction and, in addition, seats along its bottom and sides in a plane that is inclined relative to the plane of the tracks. This arrangement permits positive metallic contact on the sealing surfaces.

(*f*) *Rolling Gates.* The characteristic of a rolling gate, as shown in Fig. 19, is a cylindrical beam, spanning the opening, to which is attached the facing shown by the heavy line. The cylinder rolls on inclined tracks on the piers at the ends. A wheel of somewhat greater diameter is fitted at each end of the cylinder and rolls upon the track on the pier. These wheels have coarse pitched teeth which engage depressions or holes in the track surfaces as gears to cause equal travel of both ends. The raising force is usually applied at one end, and the force needed to raise the other end is transmitted in torsion through the cylinder. The diameter of the cylinder is ordinarily less than the damming height, and the retaining surface is completed by an apron which, when the gate is in the closed position, extends from the

cylinder to the sill and, in some cases, by an extended surface over the cylinder. Relatively large shields cover the recesses at the ends and carry the end seals.

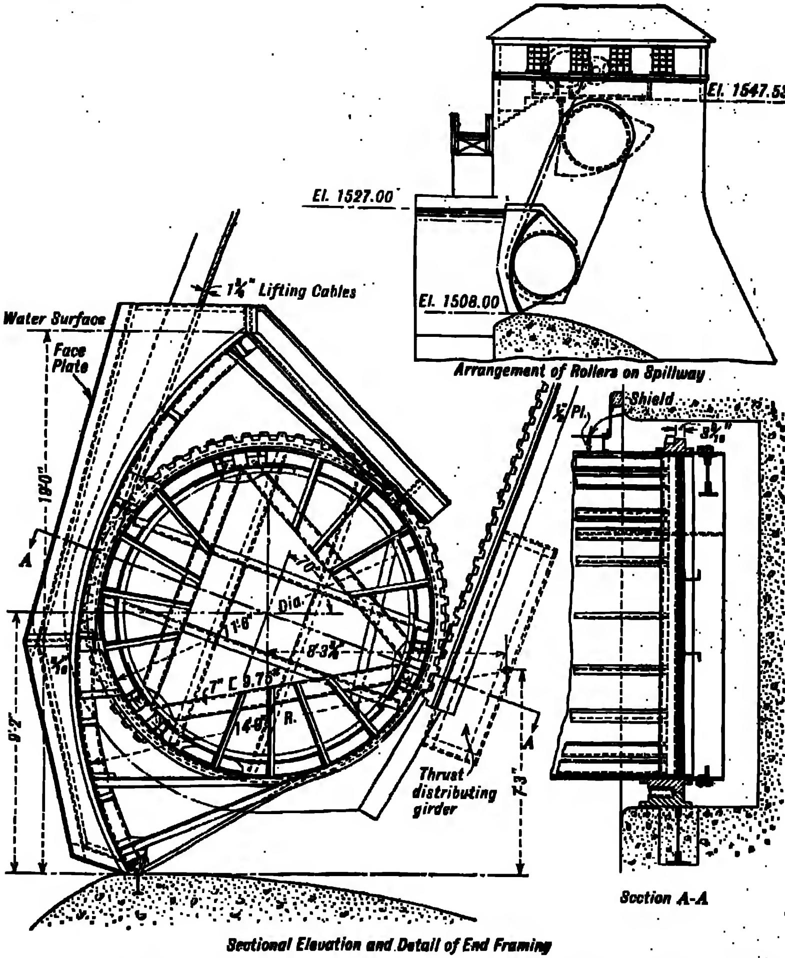

FIG. 19. Rolling gate at Long Lake Dam. (*Eng. Record*, Vol. 70, p. 322.)

The cylindrical structure is built up of steel plates with longitudinal stiffeners attached to the inside surface. Diaphragm frames are located at intervals, and solid web diaphragms are usually placed at the wheel and chain drum locations. Aprons shaped to give the most favorable water reactions when raising or lowering are constructed of steel plates with suitable bracing and stiffeners. Wheels, tracks, and chain drums are generally made of cast steel.

Rolling gates have been mounted so as to rotate below the normal closed position and have been fitted with movable flaps on the top of the cylinder to pass ice or drift from the surface of the reservoir.

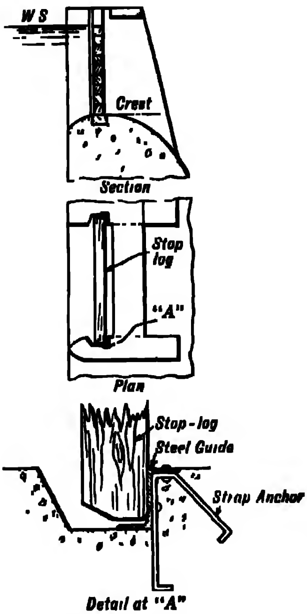

FIG. 20. Typical stoplog layout.

(g) *Cylinder Gates.* A cylinder gate consists of a steel cylinder, open at the top and bottom and having balanced water pressure on the inside or outside surfaces. Typical installations are shown in Figs. 40 and 41 of Chapter 27 and are more fully described in that chapter.

(h) *Stoplogs.* Stoplogs are vertical layers of loose timbers or steel members spanning an opening between piers or abutments and supported at each end in grooves. Typical stoplog layouts are shown in Fig. 20 of this chapter and in Figs. 64 and 65 of Chapter 27. Stoplogs, which are removed one by one as the need for increased discharge occurs, are the simplest form of crest gates. The chief objection to their use is the difficulty of installing and removing them.

For a special arrangement for installing and removing stoplogs, see Figs. 65 and 66 of Chapter 27.

Stoplogs are usually removed and installed by hand. Winches with special grapple hooks have been used for removal. They are usually made of wood for small openings, but built-up steel members have been used for the larger openings.

(i) *Needles.* Needles consist of a row of slightly inclined timbers supported at the top by a bridge or beam and at the bottom by a sill on the crest, as shown in Fig. 21. The needles are placed one by one, by extending them horizontally over the pond and allowing them to tip into the current until the lower end swings down in contact with the concrete near the sill. They are then drawn slightly upward until the lower end rests on the sill and are rolled sidewise into place against those needles already installed. They are removed by lifting each from its seat and hauling it out bodily. Each should be provided with a hole near the top through which to pass an anchor rope. The purpose of this rope is to hold the needle in case it gets away from the operators when being handled. As needles are invariably installed and removed by hand they should not be too

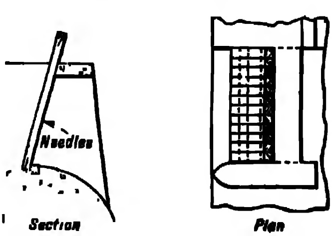

FIG. 21. Typical needle layout.

large. Timbers 6 by 6 in. and 20 ft long have been used. This type of control is not common.

(*j*) *Operation of Crest Gates.* Crest gates are lifted by hoists which vary from the ordinary rack-and-pinion geared hoists operated by hand or motor for small slide gates to very complicated hoists for large gates. A relatively small rate of power input is required because the rate of travel is slow.

Crest gates are operated either by individual hoists permanently mounted on the bridge or by movable hoists arranged to travel upon the bridge and to serve a number of gates. Choice between these two arrangements for operation is influenced mainly by the cost of the initial installation and of maintenance and the speed and frequency of operation required.

When individual hoists are used on wheeled or taintor gates, the lifting force is applied to both ends by separate hoisting devices. These devices must be tied together to insure equal travel. For short spans, the tie is usually mechanical and one motor is frequently used to drive both hoists. For long spans, electrical synchronization of the driving motors is sometimes used.

The most common form of traveling hoist for the operation of roller-bearing and fixed-roller gates is the gantry crane. The points of suspension are placed high enough and the framing of the crane is arranged so that a gate can be raised to clear the tops of the piers and carried across the bridge to other gate bays or to a work bay outside the range of gate bays.

The lift is usually effected by means of wire rope falls. Two sets of falls are used in order to apply the lift as equally as possible to both ends of the gate. Means should be provided for conveniently adjusting the lines to assure a level pick-up.

For a more detailed account of gate hoists see description of Figs. 48 through 63 in Chapter 27.

(*k*) *Force Required to Operate Crest Gates.* The force required to raise crest gates must be sufficient to overcome (1) the weight of the movable parts, (2) the friction of sliding, rolling, and pivoted bearings under load, (3) the friction of seals, and (4) incidental loads such as the accumulation of ice or sediment, with consideration given to the buoyancy of the gate, if any.

The theory of the force required for the various gates is given in Section 20 of Chapter 27. A detailed account of the capacity and efficiency of hoists for operating the crest gates is given in Section 21 of the same chapter.

(*l*) *Ice Troubles at Crest Gates.* Ice must be prevented from forming on crest gates in order to insure their being in operating condition when needed. If neglected in cold climates, ice will form in great quantities below leaky gates, and the entire upstream face of exposed steel gates will be coated with ice several feet thick. The ice which forms on the face of the gates may be too heavy for the hoist to lift and, moreover, the ice will adhere to the piers and sills. The operation of removing large quantities of ice from the gates is both slow and expensive. Aside from the great danger of possible unsuccessful operation of frozen gates when badly needed to pass sudden floods,

it will be found economical to make provisions to prevent the ice from forming if the gates must be operated during the winter season.

Three methods are used successfully to prevent freezing of crest gates. These are (1) heating by steam, (2) heating by electricity, and (3) the provision of air jets to circulate the headwater.

For the first method, the downstream face of the gate is housed in, usually with two layers of 1-in. sheeting with building paper between, and steam coils are located close to the skin plate of the gate and also imbedded in the concrete near the gate seals. Steam must be supplied continuously during cold weather because, if the pipes are allowed to cool, condensation may freeze them solid when steam is first turned on.

For the second method, space heaters have been substituted for the steam pipes inside the housing, with equally effective results.

In the third method of preventing ice formation, compressed air, released from the pipes near the bottom of the gate, creates a circulation which draws the warmer water from the bottom of the pond to the surface. This method not only will prevent the formation of ice on the face of the gate and the piers but also will keep an area of open water above the gate.

The ice must be kept clear of the upstream face of the gate, not only to permit operation but also to prevent thrust from the ice sheet against the gates. This thrust has been known to damage the gates to a considerable extent. Care should be taken not to thaw the ice away from the projecting nose of the piers, which should take the ice thrust.

Flashboards require constant attention throughout the winter. Unless the ice sheet is kept cut back from the upstream face, the thrust will cause failure. If the flashboards leak or if water is allowed to trickle over them, large accumulations on the downstream face may prevent temporary flashboards from bending over as desired or permanent flashboards from being removed. The compressed-air installation previously described has been used also for flashboard.

12. Sluices. Sluices for reservoir outlets may be divided into two classes according to the kind of service required of them. The first class serves those outlets which are operated occasionally or periodically and which are ordinarily operated with the opening fully closed or fully open. The second class serves to maintain a closely regulated outflow through a restricted opening. With low heads, all types of sluice gate are used for both classes of service. For higher heads, close regulation is effected through valves designed especially for that purpose.

The performance of sluice gates is influenced by the design of the sluice as well as the design of the gate. An abrupt change in the relation of the velocity and static heads takes place where the sluice is restricted by the gate. This causes disturbances in the sluice and frequently produces negative pressures and cavitation in the sluice below the gate. Improperly shaped entrances may cause disturbances which persist through the sluice, and irregu-

larities in the surface of the sluice or gate frame may produce harmful effects. These effects may be minimized by proper design of the water passages.

The entrance to the sluice should be shaped as nearly as possible like that of the standard orifice, and the area of the sluice should be reduced gradually between the entrance and the control. It is now generally conceded that the sluice, from the control to the outlet, should be the same section as the control. Sluices have been constructed with increasing section below the control for the purpose of regaining some of the velocity head and increasing the discharge. However, a large percentage of increase in discharge can be obtained

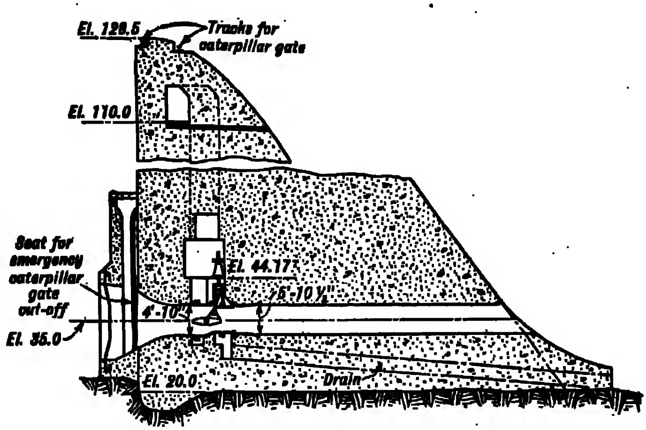

FIG. 22. Sturgeon pool development. Cross-section through discharge valves, United Hudson Electric Corp.

by this means only for very low heads, and the practice is objectionable on the grounds that it increases the vacuum and hence the erosion.

A change of section will invariably be necessary at the control. Consequently, if the control is not placed at the extreme lower end of the sluice, the concrete below the control must be well protected by a cast-iron or steel lining. The lining must be securely anchored to the concrete, and the concrete behind the lining must be drained to prevent rupture due to the pressure of seepage from headwater. Drainage may be effected by holes through the lining or by porous drains placed in the concrete behind the lining.

If the tailwater below the dam has considerable depth, the maximum capacity of the sluice is obtained when it is placed just below the tailwater surface. However, in such cases, the sluice control is not accessible for repairs. If the sluice is placed above tailwater, some head is sacrificed and leakage water will freeze in cold climates. If the tailwater fluctuates sufficiently, the sluice may be placed below the water surface corresponding to the river discharge that occurs when the sluice is ordinarily operated and will then be above water surface most of the time. Trouble from freezing can be

eliminated by providing, at the outlet of the sluice, a temporary cover which may be removed or washed out when the sluice control is opened.

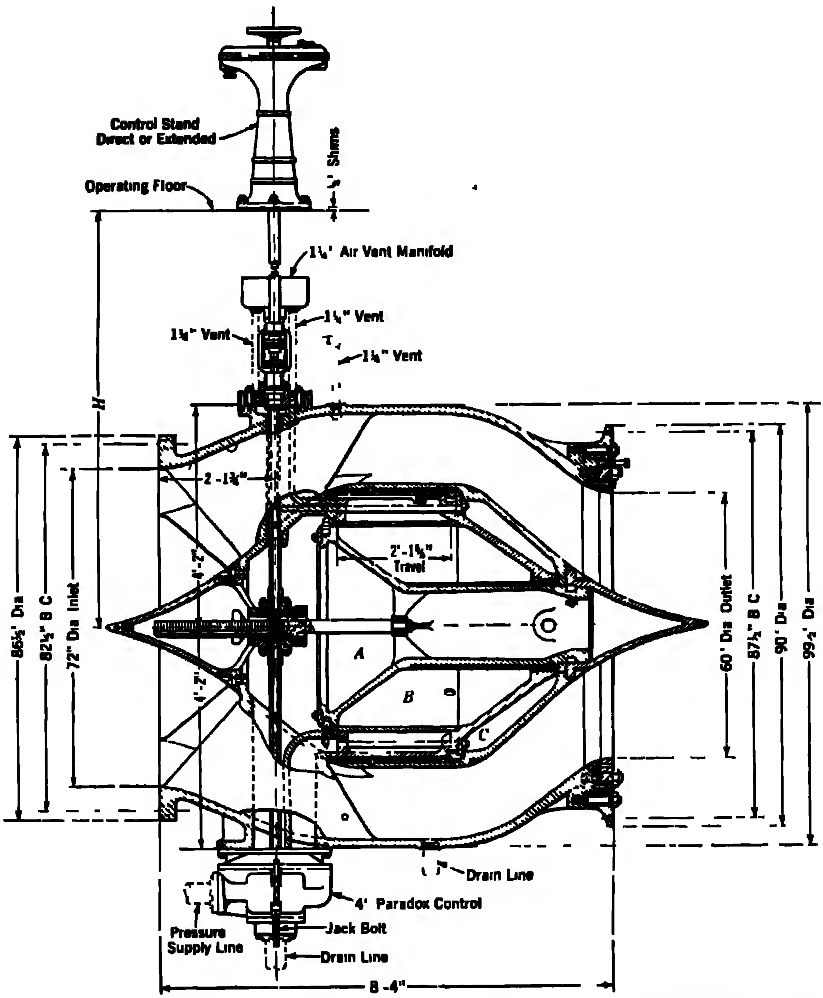

FIG. 23. Interior differential needle valve. ("Dams and Control Works," by U. S. Dept. Interior, Bureau of Reclamation.)

Figure 22 shows a typical sluice provided with a butterfly valve.

Gates of this type are generally used in masonry dams as guard gates in sluices that are normally controlled by pressure outlet valves.

The entrance to the sluice is usually protected by heavy rack bars, as shown in Fig. 22, having a clear opening equal to about one third the smallest

dimension of the full opening of the sluice control. However, some of the logs and trees which are stopped by the racks will pass part way through them and may interfere with the closure of the sluice if the racks are too close to the control. Therefore the racks should be located 20 to 30 ft above the control, depending upon the length of expected logs and trees. Racks are sometimes omitted, particularly for very deep pools.

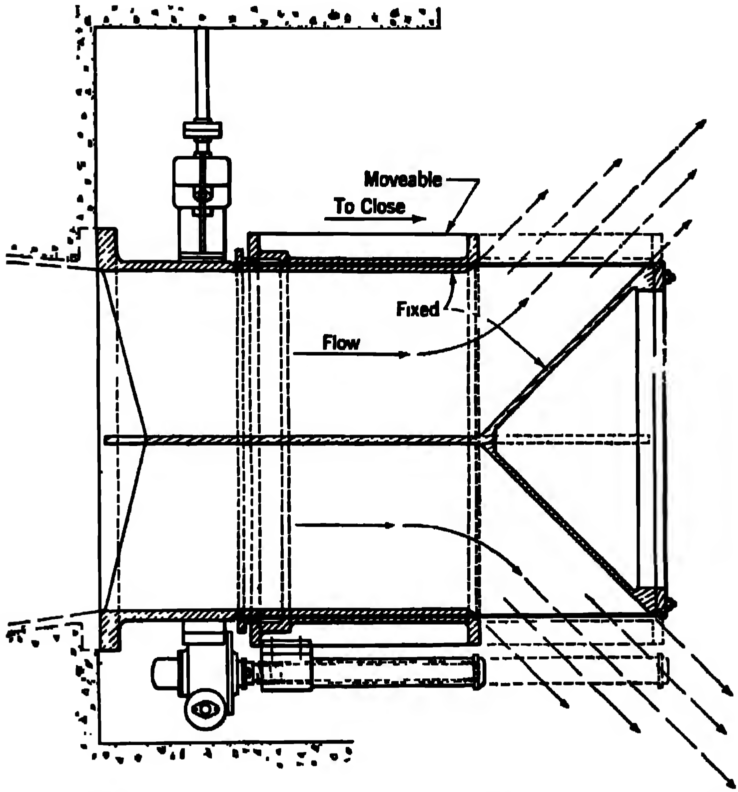

FIG. 24 Howell-Bunger valve (Courtesy S Morgan Smith Co., York, Pa.)

Provision should be made for closing the upper end of the sluice to facilitate removal of the control for repairs. Stoplogs are used for this purpose in low dams. For high dams, a seat is provided to receive a bulkhead or gate, which is lowered from the top of the dam.

A duplicate or guard gate is used where it would be very difficult, on account of the great depth of water, to plug the upper end of the sluice for repairs to the control or lining. The guard gate, being operated very infrequently, is not likely to be damaged.

As the inside of the dam is very damp, geared hoists, especially if motor-controlled, require constant care to keep them in good condition. Oil-pressure cylinder hoists are most adaptable to sluice gates and valves inside the dam, as far as cost of maintenance is concerned.

If the control is located within or at the upper end of the sluice, butterfly valves for the smaller openings (Figs. 34, 35, 42, and 43 of Chapter 27), plain metal sliding gates for medium openings (Fig. 24 of Chapter 27), and wheeled gates or caterpillar gates for the larger openings are usually adopted. Air must be introduced immediately below the control point to avoid negative pressures.

As mentioned before, the wheeled and caterpillar gates are normally operated by cable. Such operation is feasible when the gate is located on the face of the dam with hoists supported above water surface. If the gate is located within the dam, the necessity for a stuffing box between the gate and the operating chamber requires the use of a solid stem.

If the sluice is not located in the spillway section of the dam and where structural and operational conditions permit, the ideal location for the control is at the lower end of the sluice where surplus energy of the water can be dissipated outside the structure. The entire sluice is then subject to headwater pressure, which is desirable as far as hydraulic performance is concerned but necessitates watertight lining unless full headwater pressure is considered as acting in the masonry in the vicinity of the sluice.

In such cases, a differential needle valve, Fig. 23, or a Howell-Bunger valve, Fig. 24, is usually placed at the end of the sluice.

The needle valves are arranged to operate by variation of the water pressure in interior chambers under manual control. The Howell-Bunger valve consists of a fixed cylindrical body, with a cone-shaped lower end, and a hydraulically balanced cylindrical gate. The cone-shaped outlet causes a divergence of the jet which is dissipated over a very large area with resulting reduction in scour below the outlet.

13. Weights of Gates. The following figures give the weights of several types of gate. The parameters used as abscissas contain the following factors:

W = the width of the gate in feet, clear opening.
H = the height of the gate in feet, clear opening.
h = the head of water on center line of gate in feet.
All weights are in pounds.

The data for the figures were obtained from H. G. Gerdes, P. L. Heslop, and E. B. Miller,* F. L. Boissonnault [15], and the authors.

(a) Drum Gates. Figure 25 gives weights of drum gates, including all moving and stationary parts and complete control, as compiled by F. L. Boissonnault [15].

* Compiled for the Bonneville Power Navigation Project.

(*b*) *Sliding Gates.* Figure 26 gives weights of metal sliding lift gates with hydraulic hoists, as compiled by F. L. Boissonnault [15]. The weights are based on designs of the U. S. Corps of Engineers and the Bureau of Reclama-

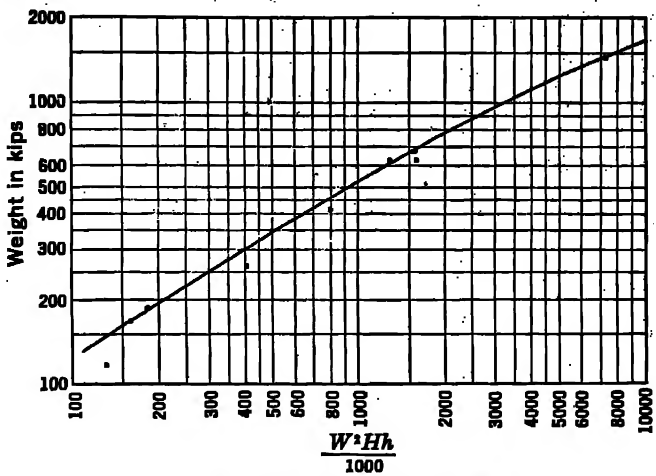

FIG. 25. Weights of drum gates.

tion and include all moving and imbedded metal with a 5-ft length of cast-iron conduit lining. No hanger, piping or valves are included.

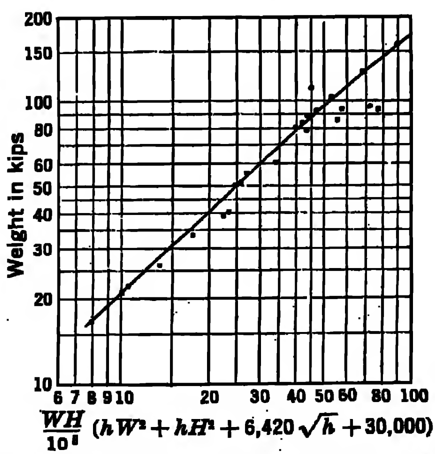

FIG. 26. Weights of sliding lift gates with hydraulic hoists.

(*c*) *Taintor Gates.* Figure 27 gives weights of taintor gates with steel face plates including anchorages and imbedded metal. The weights do not include the hoists, lifting chains, or cables. The gates are not counterweighted. All

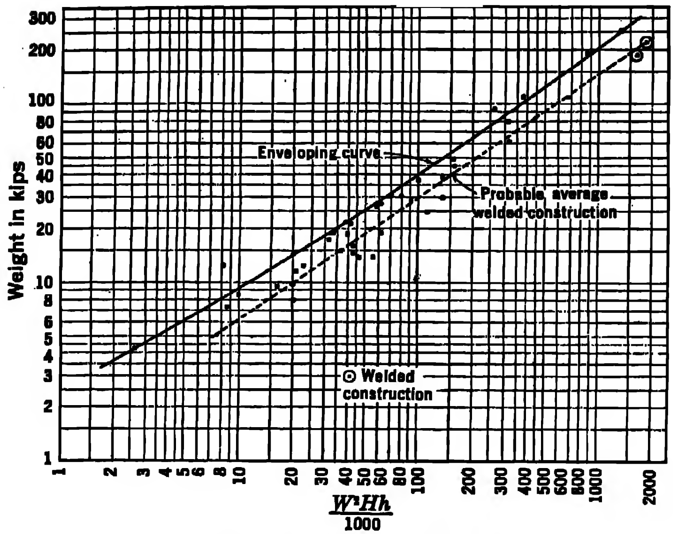

FIG. 27. Weights of taintor gates.

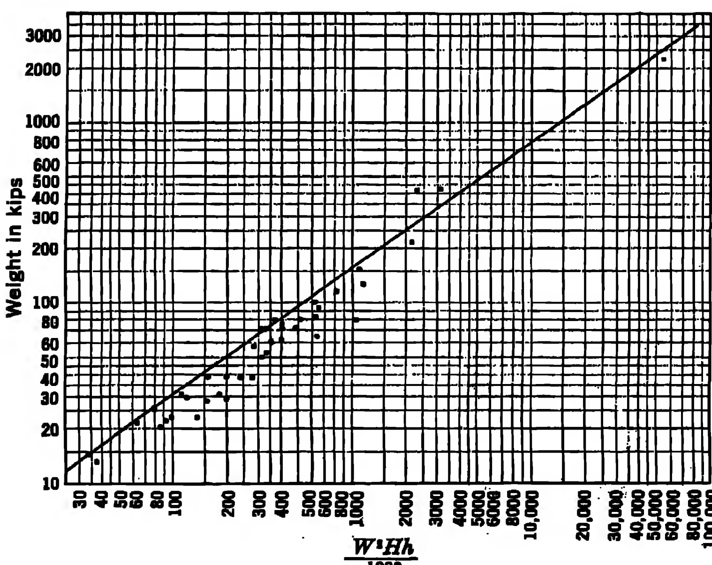

FIG. 28. Weights of wheeled and stoney gates.

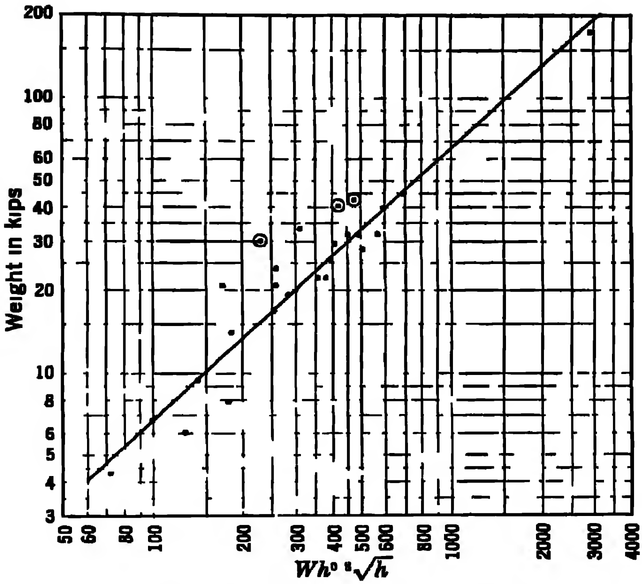

FIG. 29. Weights of caterpillar gates.

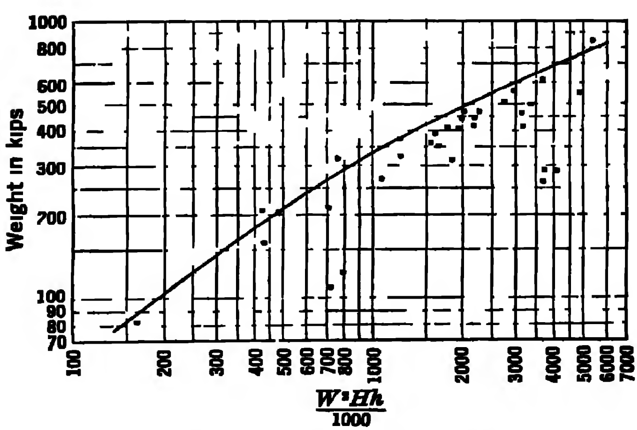

FIG. 30. Weights of rolling gates.

are of riveted construction except the two inclosed in a circle, which are of welded construction.

The dotted line in the figure is plotted to give weights which Mr. Boissonnault says are in line with probable future average welded construction as indicated by A. E. Nieberhof.*

(*d*) *Weights of Wheeled and Stoney Gates.* Figure 28 gives the weights of lift gates, mostly of the stoney type. It is believed that this figure can be used for wheeled gates.

(*e*) *Caterpillar Gates.* Figure 29 shows weights of caterpillar gates from data furnished by F. L. Boissonnault [15]. The weights include imbedded parts but not conduit lining, hoists, and cables. The encircled points have cast frames. Other points show weights of a utility line of Phillip and Davies gates, all or nearly all of which have structural-steel frames.

The authors found it impossible to derive a parameter which would cause the plotted points to approximate a straight line more closely.

(*f*) *Rolling Gates.* Figure 30 gives weights of rolling gates, including imbedded metal, but excluding hoist and chain.

14. Log Chutes. Log chutes may be required by law in places where there are logging operations on the stream. A typical chute is shown in

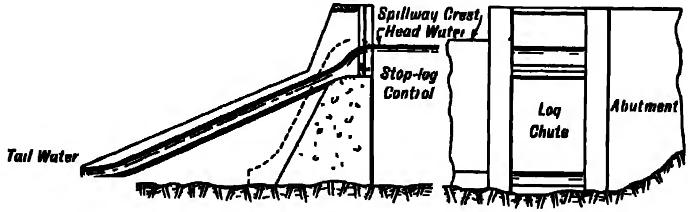

Fig. 31. A typical log chute.

Fig. 31. It consists of a depression in the crest of the dam, having a bottom elevation 2 to 4 ft below low-water surface and provided with a control through which the water passes to a smooth inclined flume. The logs may then pass down this flume to the lower pool. The gradual incline is necessary to prevent the logs from impinging, at high velocity, directly on the foundation of a shallow lower pool, with danger of their being split. The inclination of the chute varies greatly with local conditions.

The control which regulates the flow through the chute is usually of a type that lowers when more water is required. Drum gates, bear-trap dams, and stoplogs, all previously described, are frequently used for this purpose.

The chute should be made as narrow as possible in order to provide the required depth for passing logs with the minimum discharge. The depth of the water passing the control can be regulated by the operator; but the width

* *Western Construction News*, December 1943.

is usually not adjustable. The width required depends, of course, upon the size of the logs and the rate at which they must pass the dam. Some log chutes have been provided to pass only one log at a time, but chutes 30 ft wide have also been built for enormous logging operations.

15. Fishways. State law frequently requires the provision of some means for the safe passage of fish past dams. This may be accomplished either by fish ladders, consisting of an inclined trough in which the water flows from the upper to the lower pool at a velocity against which fish can easily swim, or by mechanical devices which "lift" the fish over obstructions.

The three general types of fish ladders are shown diagrammatically in Fig. 32. Sketch *A* of Fig. 32 shows the overflow type in which each baffleboard contains a notched weir over which the water spills, with a flow low enough for fish to jump. Sketch *B* is the rapids type, in which the baffleboards are shorter than the width of the trough, leaving openings at the ends for passing the flow. Sketch *C* is the sluice type, in which each baffleboard contains an opening for passing water. Openings are staggered as shown to give the water a circuitous path, thus killing as much of its velocity as possible.

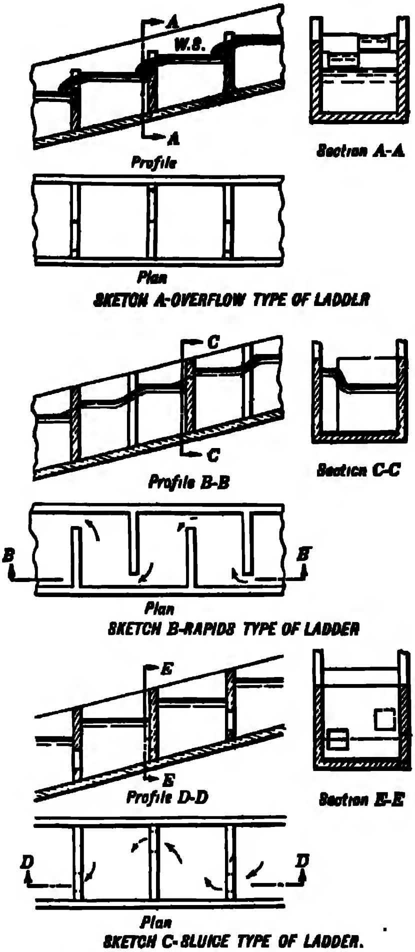

FIG. 32. Diagrammatic sketches of types of fish ladders.

Specifications for fish ladders are furnished by the states that require them and depend upon the type of fish that will use them. The main object of design is to provide as much friction as possible so that the slope of the ladder will be at the minimum for a given velocity.

One of the most successful mechanical devices for lifting fish over obstructions is the fish lock, which incorporates the essential features of a navigation lock. That is, it consists of a lock chamber, a gate-controlled entrance by which the fish enter the chamber at the lower level, a similar means of per-

mitting the fish to leave the chamber at the higher level, and a system of valves for alternately filling and draining the chamber.

The most important feature in the design of any fishway is the nature and position of its entrance. The habit of fish is to proceed upstream until halted by an obstruction and then nose their way from side to side, searching for a passage, always facing upstream. Therefore, the entrance to the lower end of the fishway must be placed near the base of the obstruction.

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CONDUITS AND POWER-HOUSES

CHAPTER 27

INTAKES

1. General. The function of the intake in a hydroelectric development is to let the water into the penstock or conduit under controlled conditions. Thus, an intake contains racks to prevent the entrance into the water passages of the plant of débris and ice large enough to injure the equipment and gates that control the flow of water through the intake. If a conduit includes an open flume or a canal, it may be advisable to have two or more intakes, for example at the entrance to a power canal and at the power plant itself.

Intakes for hydroelectric projects can be divided into two general classes: high-pressure and low-pressure intakes. High-pressure intakes are used, in general, where the drawdown is considerable, as, for instance, in a reservoir that may serve both as a storage reservoir and as the headwater of a hydroelectric development. This type of intake is more fully described in Section 10. Low-pressure intakes, which are the more common class, are used for relatively smaller drawdowns such as may be expected in the daily and weekly water-surface variations in power ponds. They are described more fully in Section 9. There is no clear line of division between the two classes of intakes.

Figure 1 shows an isometric drawing of the intake structure at the Grand Coulee development, State of Washington. Note the slightly unusual semicircular trash-rack structure.

Figure 3 shows the transition section of the penstock for this same intake beginning immediately back of the gate. This intake makes a transition from a rectangle 29.65 ft high by 15 ft wide at the gate to a circle 18 ft in diameter at a distance 34.95 ft back of the gate measured on the roof of the penstock. All losses from forebay to circular section do not exceed 15% of the velocity head at this point.

The general requirements of intakes may be listed as follows:

(*a*) *Structural Stability.* The structure must be stable even when dewatered. Low-pressure intakes are frequently an extension of, or part of, a dam and are subject to all the requirements for dams. High-pressure intakes, if in the form of an unsupported tower, must be stable against ice thrust and earthquake. (See Section 8.)

(*b*) *Limitations of Velocity.* The velocity through the racks, gates, and other passages must be confined within economic and practical limits (see Sections 5 and 7).

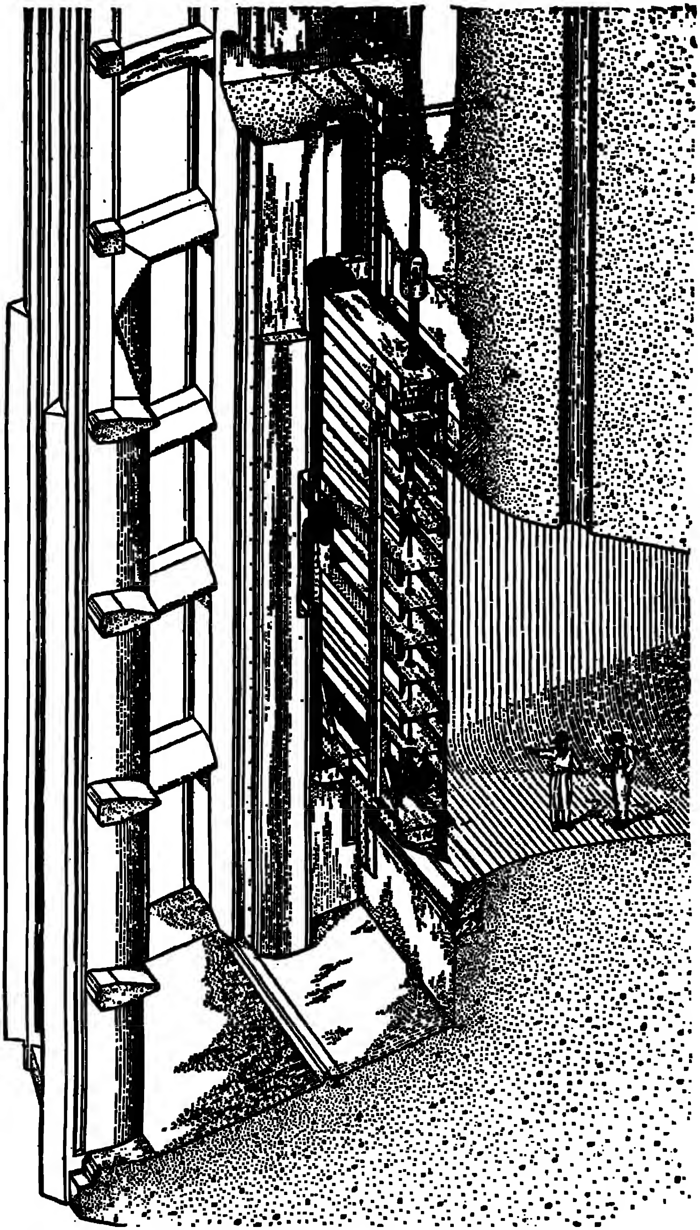


FIG. 1. Penstock intake, trash rack, and gate for the Grand Coulee, Washington, power plant. (From "Economic Principles in Design," I. A. Winter, *Trans. A.S.C.E.*, Vol. 106, p. 331, 1941.) (See also Fig. 3.)

(c) *Hydraulic Efficiency* The shape of the water passages must be such that the transformation of static head to conduit velocity is gradual and entails the smallest practical eddy losses (see Section 3).

(d) *Practicability of Operation.* The intake must be adaptable to practical operation All apparatus should be reliable and reasonably quick to operate. Delays due to repairs of conduits and turbines and to breakage of intake gates and hoists and other parts are all too frequent and must be guarded against.

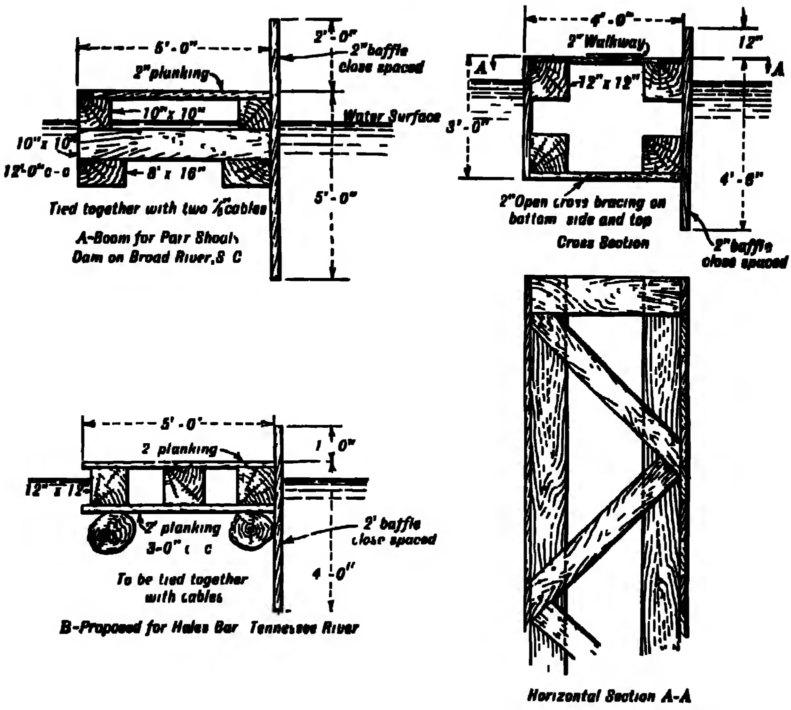


Fig. 2 Typical booms

2. Forebay. The forebay is the enlarged body of water just above the intake. It may be the pond formed by the diversion dam, or it may be the enlarged section of a canal which is spread out to accommodate the required width of the intake.

It is usually necessary to provide a deflecting device, which often consists of a boom, preferably at an angle of 30 to 45 degrees to the direction of flow, to divert ice and trash from the intake to the spillway or to a sluiceway at one end of the intake.

(a) *Concrete Baffles.* Figures 15 and 18 show a deflecting device consisting of a fixed concrete baffle wall supported on piers. This type is expensive. The baffle wall must be designed for stability against ice thrust. However,

such devices are justified in cold climates where the stream is so shallow that the top of intake must be at or near the water surface, as the ice troubles are thereby minimized. (See also Section 6.)

(*b*) *Booms.* More frequently the deflecting device consists of a floating boom, which is sometimes made from round logs fastened end to end. However, ice and débris can easily dive under such a boom, and, accordingly, booms for this purpose are frequently built up of timbers in sections from 16 to 24 ft long. These sections are fastened together to form a continuous boom.

Figure 2 shows two typical built-up timber booms. One trouble with timber booms is that they decay rapidly in water. Their life may be somewhat prolonged by painting or brush-coating with creosote the portion out of water. Thorough impregnation with creosote is not practicable, as it may make the timber so heavy that it will sink.

For these reasons some power companies have resorted to steel-pipe booms. Figure 3 shows a built-up steel-pipe trash boom as installed at the Claytor development, Virginia, by the American Gas and Electric Service Corporation. Another boom of identical design has been installed at the Winfield, W. Va., plant. At Claytor, the 1½-in. steel cable which establishes the alinement is anchored at an intermediate point in order to make the angle which it presents to the line of flow as steep as possible. The over-all length of this boom is 710 ft. The boom conducts the trash to a drop-type trash gate immediately adjoining the powerhouse. Such a boom cost approximately $27 per ft in 1946.

The tension in the boom depends on the distance the boom projects below water surface, the velocity of the water, and the sag in the boom. For practical purposes, the tension in a boom can be obtained by assuming the boom to be an arc of a circle and the pressures radial.

Let R = the radius of curvature of the boom, in feet;
α = the angle of the chord of the arc to the direction of flow;
d = the depth of the boom below water surface, in feet;
where v = the velocity of the water, in feet per second;
g = the acceleration of gravity = 32.2;
w = the weight of 1 cu ft of water in pounds = 62.5;
T = the total tension in the boom, in pounds.

Then,

$$T = \frac{wRdv^2}{g}(\sin\alpha) = 1.94Rdv^2(\sin\alpha)$$

Ample allowance should be made for the indeterminate effect of wind and the friction of flowing water on an accumulation of ice and débris against the boom, and also for the impact of this accumulation.

Although the impact of ice floes and débris on floating booms usually is not severe, allowance must be made, of course, for unusual conditions. The boom should be loosened from its anchorage after it becomes imbedded in ice, if a movement of the ice sheet is to be expected.

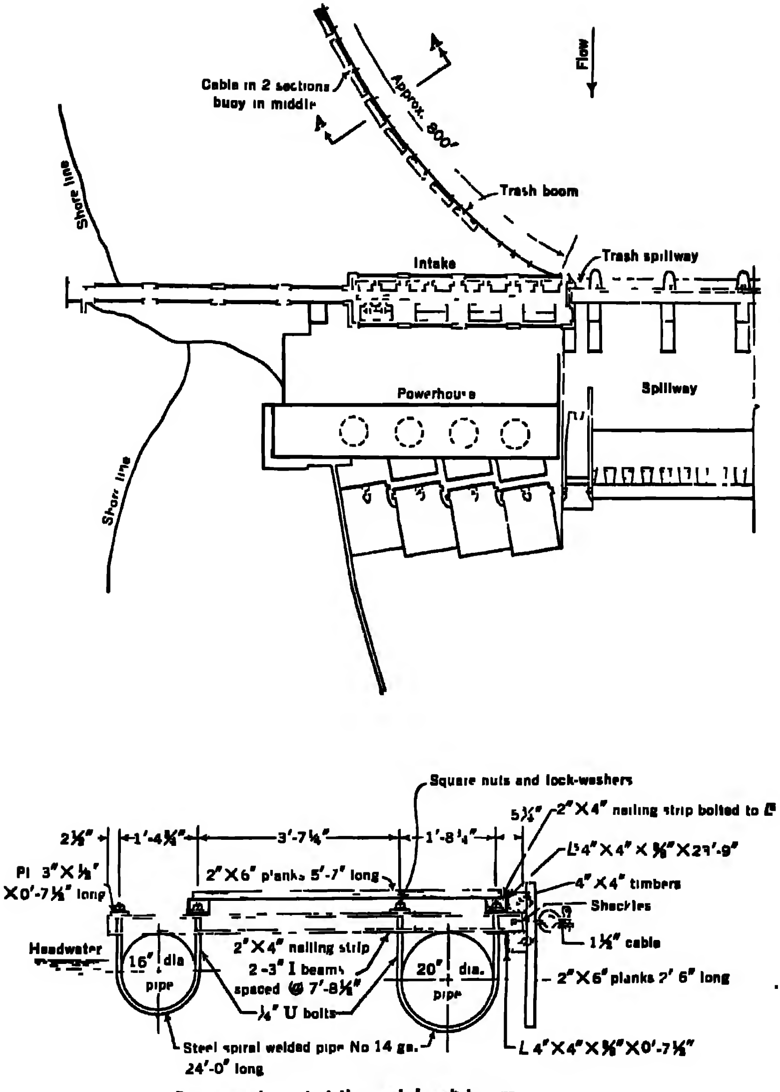


FIG. 3 Plan and section of steel-pipe trash boom. Claytor hydroelectric development of the Appalachian Electric Power Co. (American Gas and Electric Service Corp., New York, N. Y.)

Single-log booms are simply fastened at the ends by means of chains or cables, but built-up booms are usually supported on cables or chains that are continuous between anchorages. Cables are better than chains if more than one is required, because, if there is unequal stress in the cables, those having the greater stress will stretch until the tension is fairly well distributed among them, whereas chains, having less elasticity, would be likely to break under such conditions. Cables should be galvanized or well protected with paint or heavy waterproofing lubricant.

With long booms, the tension may be so great that intermediate anchorages are necessary. These can be provided by concrete or rock-filled timber cribs, or as is frequently done, cables or chains can be attached to the boom at several points and anchored to the bottom of the forebay above the boom. With the latter arrangement, an adjustment of the boom can be made by varying the length of the cables.

Floating booms should be composed of well-seasoned softwood timbers and should be painted to prevent waterlogging. The ends should be anchored so that the connection is free to rise and fall with fluctuations in the water surface. The depth to which the boom and its baffle projects beneath the water surface is usually about 3 or 4 ft.

A sluiceway at the outlet end of the boom, as shown in Fig. 17, is essential in order to pass cake ice and trash when no water is flowing over the spillway and to create a greater surface velocity toward the outlet. The sluiceway is usually provided with stoplogs or an overflow gate to regulate the depth of outflow.

3. Hydraulics of the Intake. It is of great importance in the design and construction of intakes to have the water passages as smooth as possible in order to eliminate all unnecessary losses of energy. Changes of section

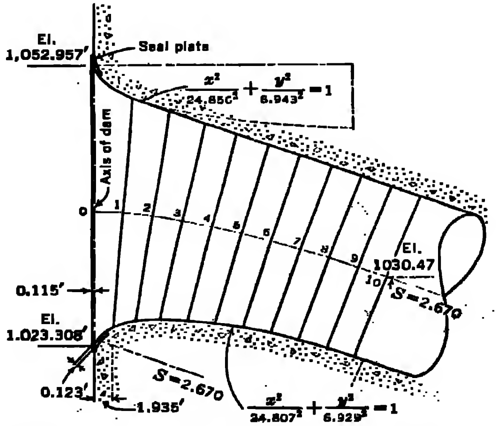


FIG. 4. Intake transition of Grand Coulee penstock. ("Economic Principles in Design," I. A. Winter, *Trans. A.S.C.E.*, Vol. 106, p. 331, 1941.)

should be made gradually, not abruptly. It will pay the engineer to devote considerable time to a study for determining the smooth gradations in curvature that will give minimum losses. Smoothness will frequently mean warped surfaces and a consequently increased cost for formwork, but very often the energy gained will pay a large return on this additional investment.

The most desirable and economical curvature for these gradations is one that makes the velocity changes plot on a straight line. That is, when the velocities at various intermediate sections between the prescribed sections are plotted as ordinates and distances as abscissas, the minimum loss will take place if the points plot on a straight line.

At the Grand Coulee power plant on the Columbia River, careful attention was given to the transition section between the intake and the penstock immediately back of the head gate (see Fig. 1, Table 1, and Fig. 4)

TABLE 1 *

AREAS AND DIMENSIONS OF SECTIONS IN TRANSITION
GRAND COULEE PLANT

Section	Dimensions at Center Line in Feet		Ellipse Axes in Feet		Area of Sections in Square Feet		Developed Distance from the Origin at Seal Plates	
	Height	Width	Vertical	Horizontal	Jet †	Actual ‡	Roof §	Floor ‖
0 ¶	29.65	15 00	Rectangle		444.7	444.7	0	0
1	23.72	15.30	11.86	0 63	356.6	356.5	5.45	3.30
2	22.09	15.60	11 05	2.21	324.5	323.7	9.25	5.60
3	20.90	15.90	10.45	3.21	305.1	303.5	12 95	7 95
4	20.05	16.20	10 03	3 99	292.5	290.5	16 10	10.70
5	19.34	16.50	9.67	4.65	284.2	280.5	19.35	13.40
6	18.82	16.80	9.41	5 35	278.2	273.0	22.57	16.10
7	18.48	17.10	9 24	6.18	275.0	267.0	25.77	18 85
8	18.25	17.40	9.13	7.05	273.0	262.3	28.80	21.70
9	18 10	17 70	9 05	8 05	271.9	257.8	31.92	24.60
10	18 00	18 00 **	Circle		271.7	254.5	34.95	27.40

* From "Economic Principles in Design," by I. A. Winter, *Trans. A.S.C.E.*, Vol. 106, p. 344, 1941.

† Normal to axis of dam.

‡ Normal to flow.

§ Origin at elevation 1052.957.

‖ Origin at elevation 1023.308.

¶ Gate seal.

** Diameter.

to the point where the regular 18-ft-diameter penstock was reached at a distance approximately 35 ft downstream from the head gate

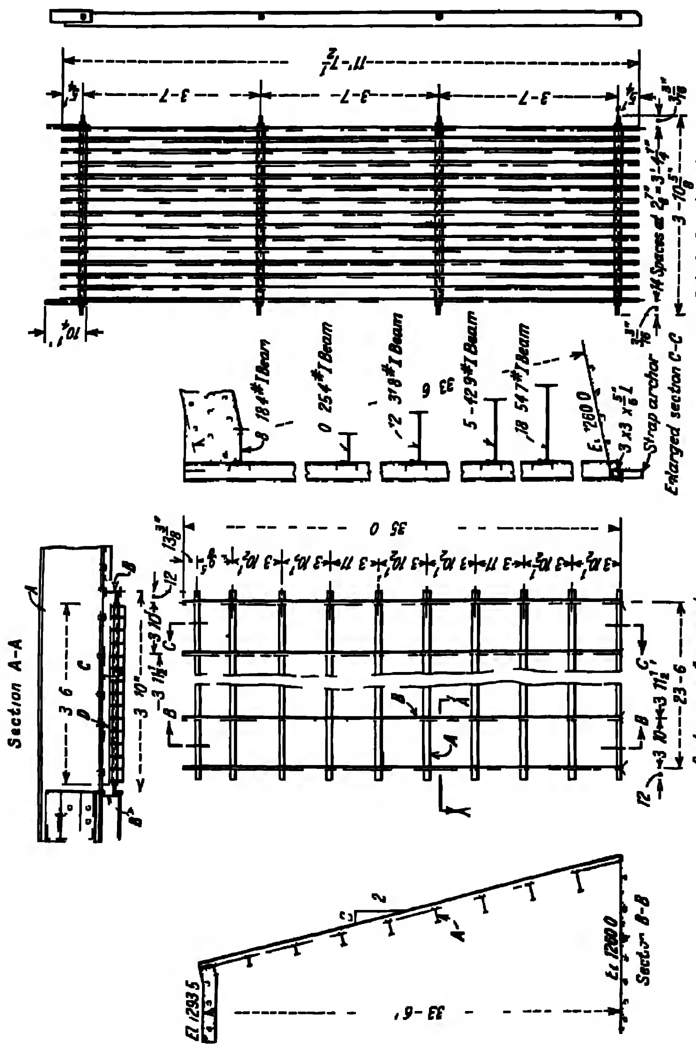


FIG 5 Trash racks at Soft Maple development Northern New York Utilities Inc

4. **Racks.** (*a*) *Inclination of Racks* Racks for the purpose of keeping out débris that might injure operating equipment are generally located ahead of the gates Formerly they were almost universally erected in a slanting position as in Fig 5, so that débris and leaves could be raked from the racks

to a platform by hand and then disposed of. The slope of the racks was often 1 vertical to ¼ or ½ horizontal. Mechanical rakes, as in Figs. 8 to 11 inclusive, have made the slope less important, and many racks are built vertical as in Fig. 7. Many engineers still prefer the racks to slant slightly.

Figure 9 shows an arrangement of the Newport News mechanical rake which facilitates the disposal of the débris through a sluice.

(*b*) *Spacing of Racks.* With small turbines, it is necessary to use close spacing of trash-rack bars (2½-in spacing being not uncommon for small units). For large units with large water passages, much wider spacing is permissible and generally desirable. However, about 5-in. spacing is generally considered the maximum desirable, as anything much larger would permit the entrance of timbers that might injure the wicket gates or runner.

It is usually wise to adopt the rack-bar spacing recommended by the turbine manufacturer. Rack bars and their supporting structures, when subject to complete clogging, are designed for very high working stresses. (See also Section 8.) A usual criterion, for temperate climates, is that rack bars and their steel supports be designed for 25% of the total head to which they might be subjected if wholly clogged with outer fiber stress limitation in the steel of 16,000 lb. Accordingly, rack bars might not fail even in case of complete stoppage.

(*c*) *Construction of Racks.* Trash racks are made of steel bars of rectangular cross-section set parallel to each other They are usually built in panels 4 or 5 ft wide and not too long for handling. Figure 4 shows details of the racks of the Soft Maple development. The main supports for the racks consist of horizontal I beams, *A*, spanning each intake bay. Attached to the supports are the inclined I beam and channel guides, *B*. The rack spacing bolts, *C*, project sufficiently to slide within the inclined guides. The racks rest on the small channels, *D*, attached to the main supports, which serve merely as a filler to make the racks project beyond the center flange of the inclined guides.

The rack bars are usually ¼ or ⅜ in. thick, the latter thickness being more frequently adopted on account of the desirability of greater stiffness in handling. They are usually 2½ to 3 in. wide. For very wide spacing of bars for large units, the bars may have to be larger in order to be structurally safe. At intervals, the bars are drilled as close to one edge as practicable, and rods ⅝ or ¾ in. in diameter are passed through them. Thimbles, obtained from short lengths of steel pipe, are threaded on the rods to act as spacers to keep the bars apart. The rods are threaded at the ends, and a section of the bars is bolted together. The edge to which the drilled holes are closest is toward the downstream side of the racks, so that the teeth of the rake can pass between the rack bars without interference and remove anything lodged on or between them.

It is now common practice to weld up frames of rack bars as in Fig. 7.

If no crane is provided for handling the racks, they are usually made in sections sufficiently light for hand removal and replacement and are built

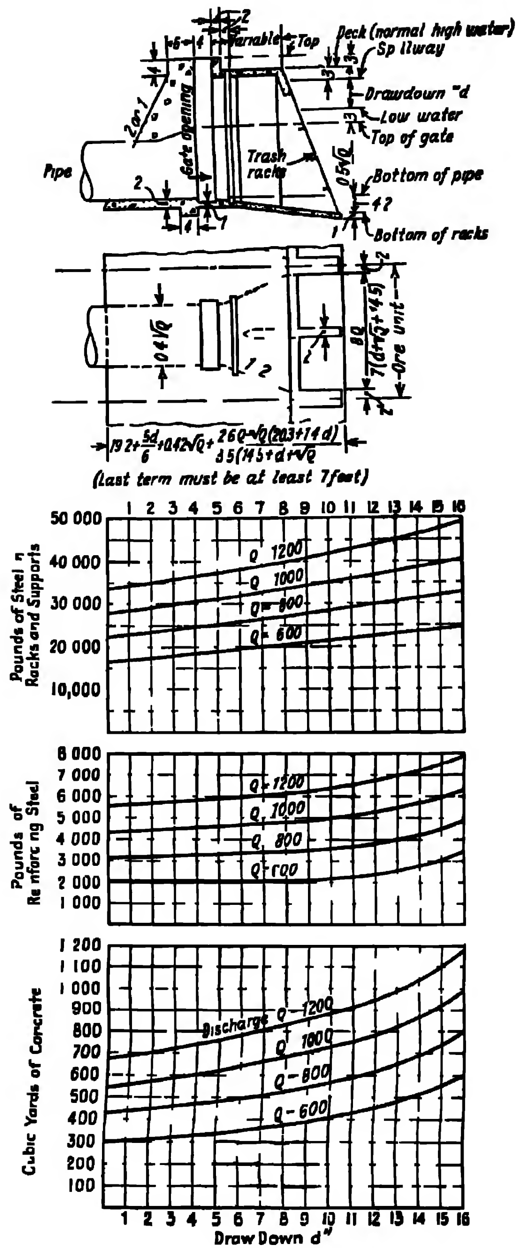


FIG 6 Quantities in one unit of intake

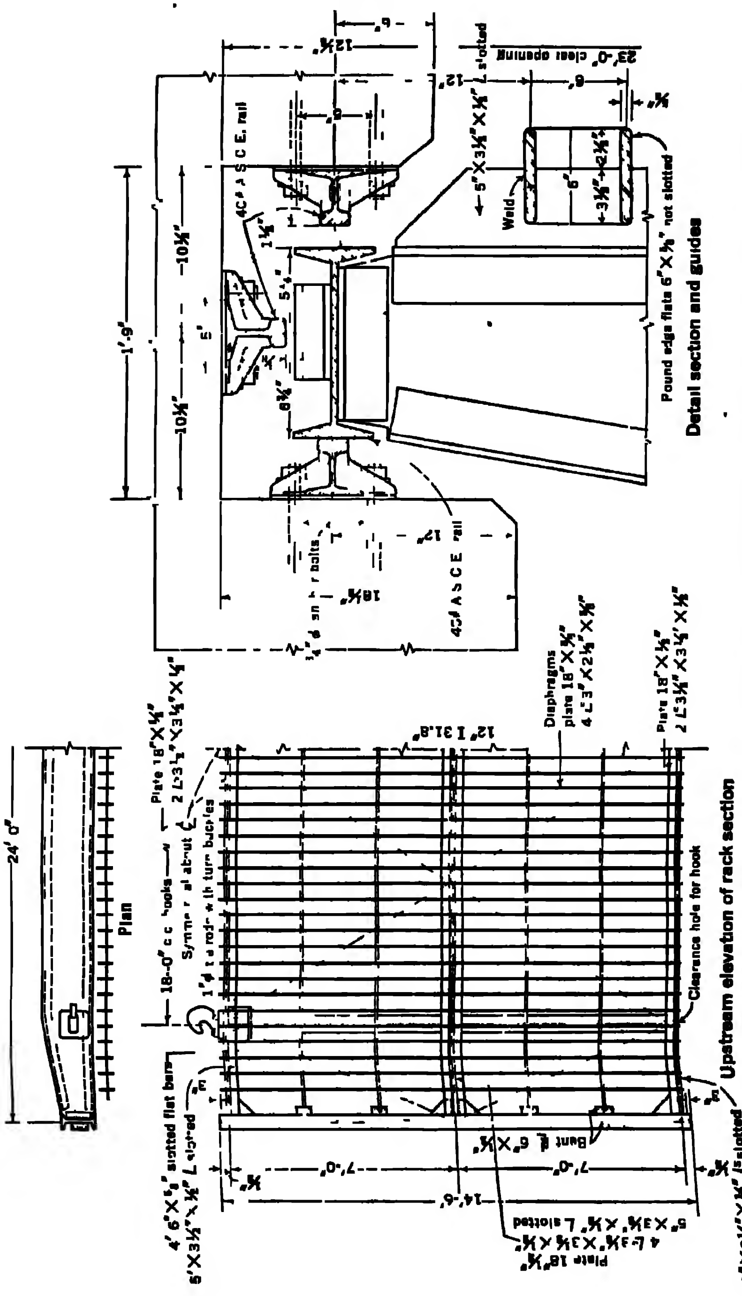


FIG. 7. Section of welded trash rack Conowingo hydroelectric development Maryland (Philadelphia Electric Co.)

up before being lowered into place. However, where the top sections are to be removed during times of anchor ice and a crane is provided for lifting them, they are sometimes made in very large sections.

It is essential that the sections be stiff enough to prevent springing during handling. This feature is particularly necessary if the top sections are to be removed when anchor ice is running, as previously discussed.

No projection of any kind which will engage the rake teeth and interfere with raking should be allowed on the racks or the supports. If the racks are

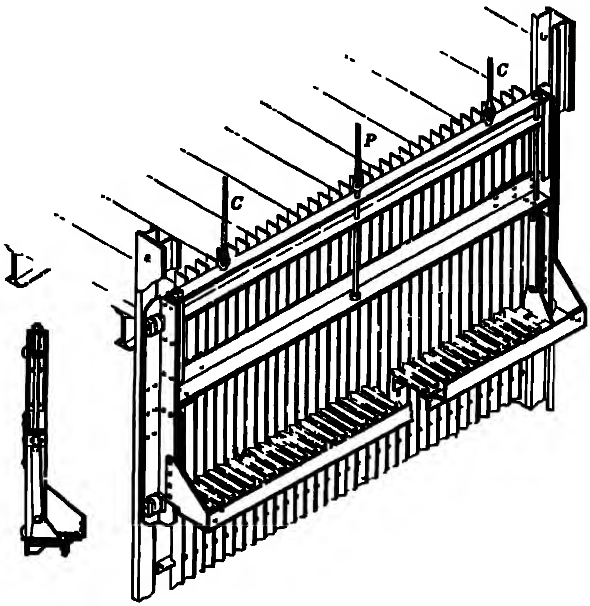


FIG. 8 Mechanical rake (Newport News Shipbuilding and Dry Dock Co)

made in sections, the sections must join perfectly so that each bar of one section will line up with the corresponding bar of the section below. The rack supports are frequently designed not only as beams to support the racks but also as struts to support the side walls against water pressure. Approximate weights of rack and supports for the usual type of low-pressure intakes are given in Fig. 6.

The usual velocities of flow through racks are discussed in Section 5.

(*d*) *Raking Racks.* At small plants, hand raking of racks is practiced. On most streams it is necessary to do very little raking of racks during the greater part of the year, but in the fall, when the streams carry a great many leaves, intensive raking is frequently required. Also, at times when they are carrying a large amount of débris, raking becomes quite a problem. Streams vary greatly in this respect; on some, in order to keep the racks

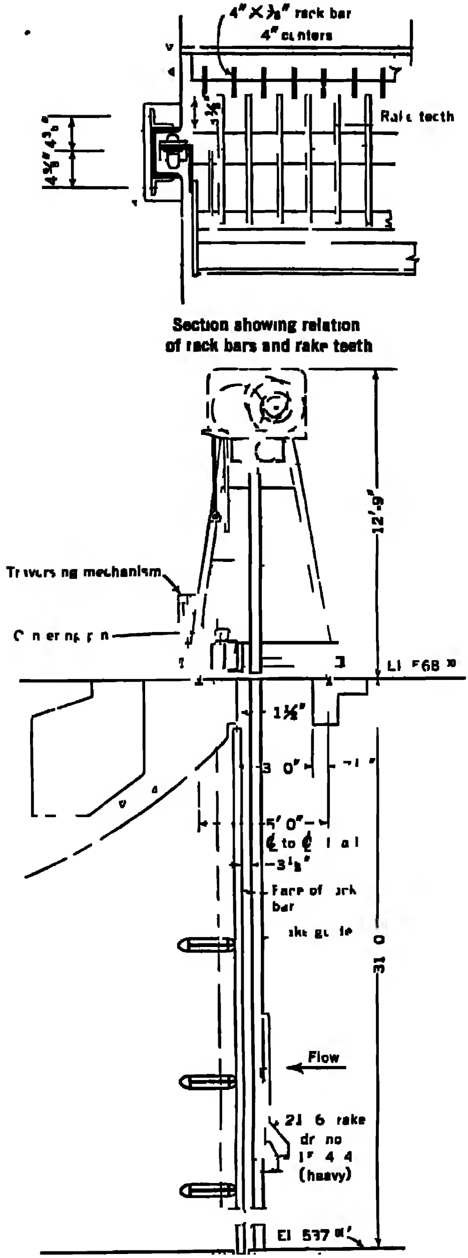


Fig. 9 Arrangement of Newport News rack rake which disposes of raked trash through a sluice. Winfield plant of Kanawha Valley Power Co.

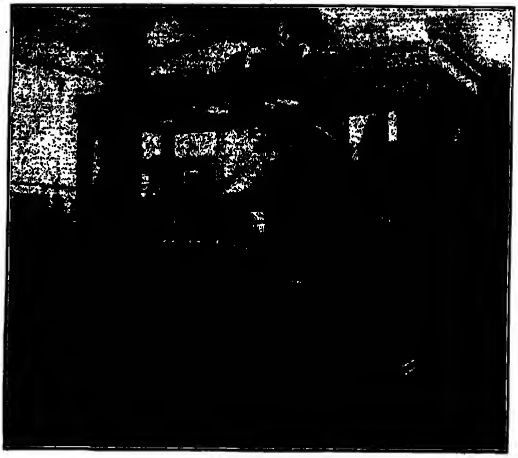


FIG. 10. Leonard trash rake and car with hopper for Ebasco International Cia. Paulista de Força e Luz, Avanhandava hydroelectric project, Brazil, S. A. (S. Morgan Smith Co.)

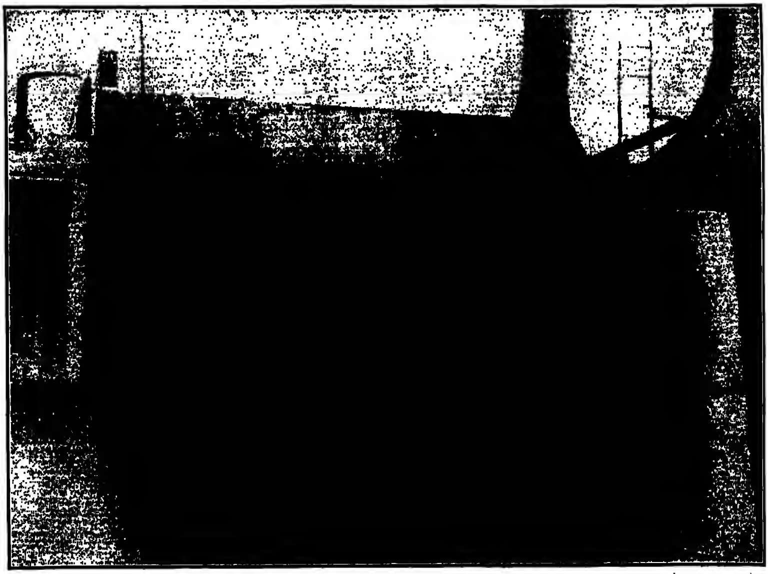


FIG. 11. Leonard trash rake, Santee Cooper Dam. (South Carolina Public Service Authority.)

sufficiently free to permit the efficient operation of the units, constant raking with considerable force is required at certain seasons of the year.

If the intake is submerged materially, there is no problem of raking.

The hand rake consists of iron or wooden tines spaced to fit between the rack bars. The handle is sometimes a steel pipe, but this makes a rather heavy tool for deep raking. A convenient type consists of an iron rake-head with tines to cover a width of rack bars of about 18 in., and a handle of selected ash about 1½ in. in diameter.

The cleaning of racks may be greatly minimized by means of a compressed-air bubbler system as described and illustrated in Section 6g. By using this bubbler system with unit at part gates and headwater elevation above the top of the racks, practically all trash may be brought to the surface and passed over into the sluicing trough, as in Figs. 16 and 17, and disposed of to tailwater.

(c) *Mechanical Rakes.* If a great deal of raking is required, it pays to install a rack-raking machine. Many of these machines are in use at various plants, and they are generally proving effective in keeping the racks free of débris under severe conditions. The wholly mechanical type of rake is preferable. Such a machine, as made by the Newport News Shipbuilding and Dry Dock Company, is shown in Fig. 8. In this figure, when the rake is being lowered from a drum hoist, tension is on the line *P*, and the teeth of the rake hang loosely with the back ends lower than the front. On the ascent, the tension is on the line *C*, which pulls the teeth up straight and causes the front end of the teeth to ride between the racks and do the raking. Illustrations of mechanical raking devices are shown in Figs. 9, 10, and 11.

5. Velocities through Racks. (*a*) *For Low-pressure Intakes.* With small units and consequent closely set rack bars, design criteria have usually limited maximum velocity through the gross cross-section area of the racks to about 2.5 ft per second with reliance on hand raking. With large units and mechanical raking, wider spacing of rack bars is permissible, and maximum velocities of 5 ft per second have been used successfully in low-pressure intakes. For instance, at Wheeler Dam (T.V.A.) there is a velocity of 5 ft per second through the gross area of the racks under full-load conditions. Under this condition the flow was 9400 sec-ft with a gross loss of 0.1 ft [1]. Gross losses through racks are generally 0.1 ft to 0.5 ft and in the judgment of the authors should not exceed the latter amount. (See Section 6, Chapter 8.)

(*b*) *For High-pressure Intakes.* As in Figs. 41, 42, and 43, considerably higher velocities may be advisedly used through the racks of high-pressure intakes. It is seldom necessary to rake the racks of such intakes, and over-all economy rather than convenience may determine the velocity to be used in raking racks. Many high-pressure intakes operate successfully at maximum velocities of 10 or 12 ft per second through the gross area of the racks.

6. Prevention of Ice Troubles at Intakes. (*a*) *General.* In the design of intakes located in cold climates, it is often necessary to adopt special precautions to prevent ice from interfering with the operation of the plant (see also Section 9*f*).

Figure 13 shows a type of layout that is likely to invite trouble from floating ice, and Fig. 14 shows a general layout that minimizes this problem. A deep and quiet pond that will permit the early formation of an ice sheet is desirable. If there is also a deep intake (the top of which is, say, 40 ft below water surface), trouble from ice even in the coldest climate will be practically eliminated.

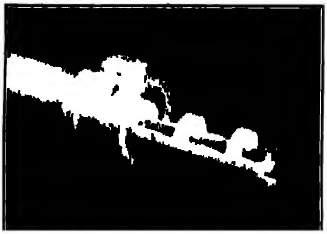


FIG. 12. Hynes electric straight tubular heater used for keeping gate and rack guides free from ice. Shown with heating pile partially removed from casing. (Lee P Hynes)

The operator has to contend with three distinct forms of ice, namely, sheet ice, frazil ice, and anchor ice.

(*b*) *Sheet Ice.* The small ice crystals that form when the temperature of the water at the surface falls to the freezing point accumulate to form an ice sheet if the current of the water does not disperse them. In swiftly moving water, the ice sheet forms at the sides, where the velocity is relatively low, and gradually builds out until the whole surface is covered. If the velocity is too high, or if fluctuations of the water surface loosen the sheet at the sides as fast as it forms, the surface will not become covered even in the coldest climates.

When the ice sheet melts, it breaks up into large cakes that float down to the intake and must be disposed of through ice chutes or over the dam. Ice chutes at the lower end of canals are essential to get rid of floating ice.

(*c*) *Frazil Ice.* The small ice crystals that form at the surface or in rapids and are not held back in the formation of an ice sheet appear at the intake and are known as "needle" or "frazil" ice In the eddying of the current the crystals clot together loosely in soft masses termed "slush" ice, which float at various depths. This ice has a tendency to adhere tightly to all structures having a temperature equal to or less than the freezing point of water The adherence of the ice to the racks and the turbines may completely plug all openings within a short time after the ice starts to run. Rakes will not effectively remove this ice from the racks.

(*d*) *Anchor Ice.* Anchor ice consists of small ice needles which form at the bottom of fairly quiet, shallow bodies of open water or water covered by a thin transparent ice sheet It appears usually on cold, clear nights and becomes loosened and floats away during the early hours of the day. In action and appearance it is very similar to frazil ice, previously described.

Except in rare cases, frazil and anchor ice occur only when there is no ice cover on the river, pond, or canal. Consequently, when the powerhouse

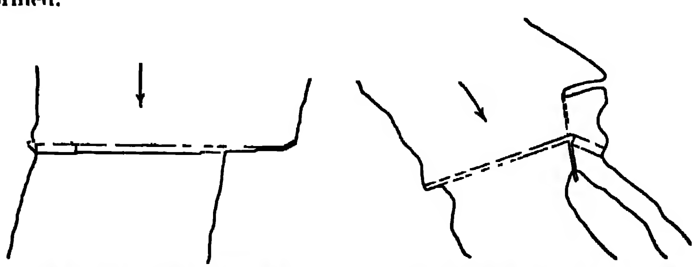
is located in the dam and a pond of considerable size lies above the dam, there is seldom serious trouble from frazil or anchor ice. Similarly, little or no trouble need be expected when the approach to the powerhouse intake is through a long, deep canal at a velocity low enough to permit the ice sheet to form readily. However, at practically all plants in cold climates, there will be some formation of frazil and anchor ice during the late fall or early winter before the water has a chance to freeze over.

Low head, shallow intake, absence of boom or skimmer wall attract ice to intakes

Funneling of river towards powerhouse tends to choke intake with ice

FIG. 13. General layout inviting trouble from floating ice. (From P. E. Gisiger, Pennsylvania Water and Power Co.)

On the other hand, when conditions are such that the water is brought to or near the intake at a relatively high velocity, which tends to prevent the formation of an ice sheet, a great deal of frazil and anchor ice is likely to be formed.



Large size pool, deep intake, are adequate protection

Moderate size pool, shallow intake requires skimmer wall or substantial boom

FIG. 14. General layout minimizing trouble from floating ice. (From P. E. Gisiger, Pennsylvania Water and Power Co.)

The greatest source of trouble is frazil and anchor ice. Fortunately, these formations are very loosely held together and possess little or none of the structural strength of sheet ice. Accordingly, if they can be prevented from sticking to the surfaces of the racks and turbines and thus causing clogging, they may be passed through the plant without making trouble.

(c) Adherence to Surfaces. It has already been stated that frazil and anchor ice will adhere to surfaces that are at or below the freezing tempera-

ture of water. If such surfaces can be kept only a very small fraction of a degree warmer than freezing temperature, no difficulty will be experienced. The greatest trouble occurs at the racks, because the turbine parts are usually kept at a temperature above freezing by conduction from the warm air surrounding the generator.

The three general methods of preventing trouble from frazil and anchor ice at the racks are: (1) heating the racks, (2) diverting the ice with compressed-air jets, and (3) removing the rack during ice runs.

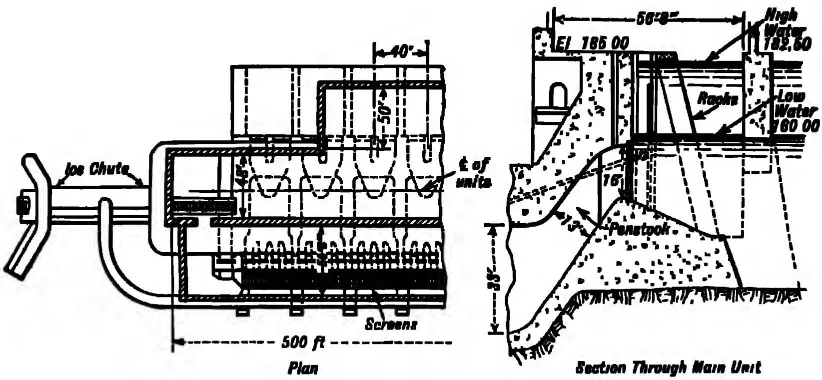


FIG. 15 Outside baffle wall at Holtwood plant (McCall's Ferry), Susquehanna River, Pa., used to divert ice. (From *Eng. Record*, Vol 56, p 319)

(*f*) *Heating of Racks.* For the heating of the racks and other steel surfaces to which ice may adhere, the entire structure above water surface must be housed in a baffle wall ahead of the racks. This wall, which extends slightly below water surface, as in Fig. 15, serves to exclude the outside air. Warm air is then introduced inside the housing, and this warms the upper end of the rack bars, which should extend some distance above water surface into the housing. The rack bars should be continuous from top to bottom, and by conduction the heat is transmitted to the lower part of the racks. This method of heating is not entirely effective when the conditions are severe.

At the La Turque development above Shawenegan Falls on the St. Maurice River, Quebec, the racks are housed in and coils of steam pipes are placed in contact with their projecting tops. This device has practically solved the problem at that plant.

Heating the rack bars by electricity has been practiced to a considerable extent in Norway and Sweden, where very extreme cold weather is experienced. It is said to be more economical and more convenient than steam [2].

The necessity for heating of racks arises from the exposure of the top of the racks to the atmosphere. Accordingly, the designer should strive, in

a cold climate, to insure the submergence of the racks at all times and thus minimize the need for heating

It is desirable to have a 4- to 8-in pipe inserted in the concrete just back of the guides in which the rack bars are inserted, as in the more common practice in installing gate guides in cold climates. Electric heating piles are

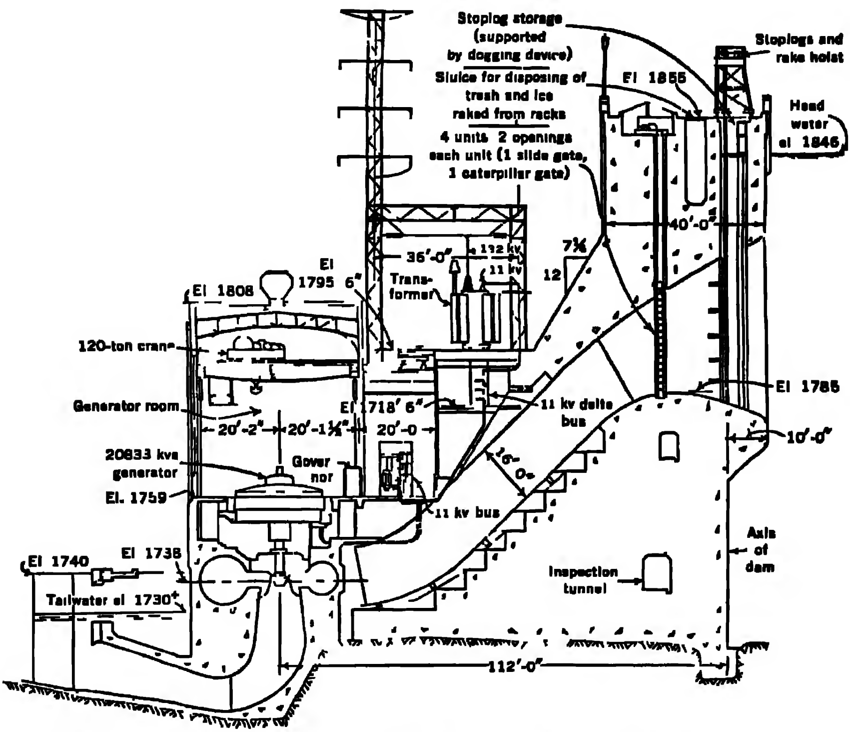


FIG 16 Section through intake and powerhouse, Claytor hydroelectric development, Virginia (From American Gas and Electric Service Co)

installed similar to the pile shown for gate guide installations in Fig 12 By this means the designer insures that the racks or gates will not be frozen in the guides if it should be necessary to remove them quickly in case of a run of frazil ice as in Figs 19 and 20 Figures 16 and 17 show power plants provided with sluices for disposing of ice and trash

At Grand Coulee Dam, where the temperature falls to 28 degrees Fahrenheit below zero, intensive investigations and tests were made by the U. S. Bureau of Reclamation to determine the most desirable methods of de-icing. As a result, a system of heating pier plates and seats for drum gates by means of electric currents induced in the steel plates themselves makes it possible

to use only a relatively low temperature in electric cables and reduces danger of overheating the adjacent concrete.

For heating the bars, either single bars or groups of bars are connected in series and the current is passed through them. It has been found that,

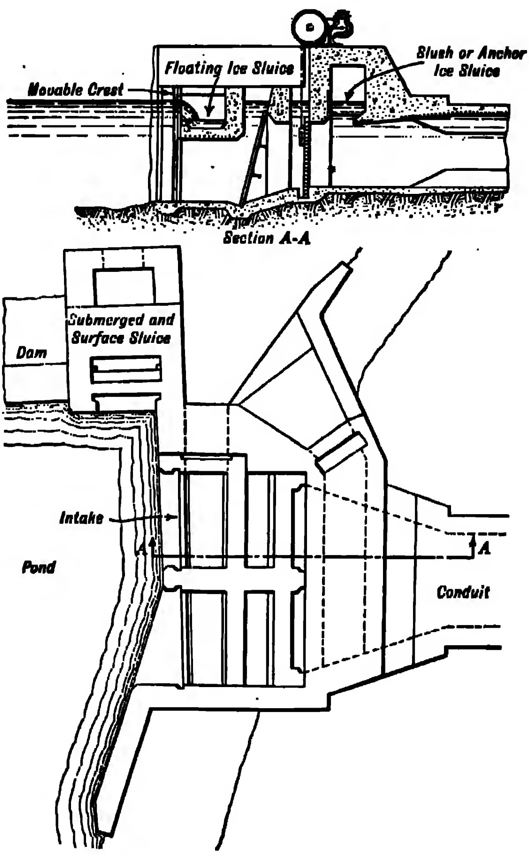


FIG. 17. Arrangement of ice sluice at Plant 3, Deerfield River. (New England Power Co.) (From *Eng. Record,* Vol. 67, p. 153.)

for bars having a cross-section area of 0.6 to 1.0 sq in., sufficient heating is obtained with 250 to 300 amperes per bar. In the walls of the intakes and of the ice chutes, electrical heating units are sometimes installed to heat the concrete surfaces so that ice will not stick to them.

(*g*) *Bubbler System.* Bubbler systems that can also be used for the removal of trash from the racks have been found very effective in dealing with ice troubles. Compressed air under pressure somewhat greater than enough to balance the depth of water is released from a perforated pipe in front of

the intake racks. The bubbling air causes the frazil, anchor, and cake ice to rise and go over the top of the rack into the sluice box provided for the purpose, from which it can be sluiced to the tailrace. The proper distance from the foot of the racks to the location of the air pipe depends on the velocity of the water, as it is essential that the air reach the surface before coming in contact with the racks, and also that it reach the surface near enough to the racks for the ice to be drawn into the sluice.

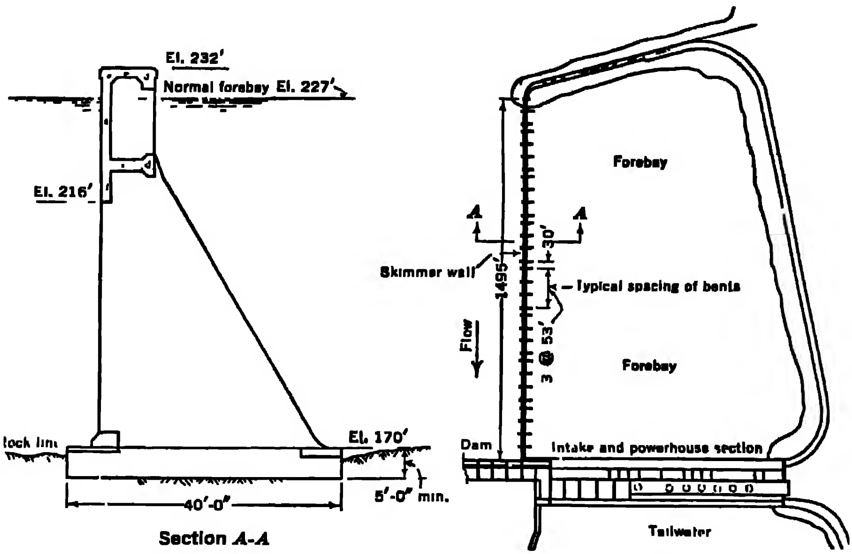


FIG. 18 Forebay and baffle walls for ice at Safe Harbor hydroelectric development, Pennsylvania.

At the Rocky River hydroelectric plant in Connecticut, a bubbler system has satisfactorily kept the intake free of ice and trash for the past twenty years. The system at the intake is shown in Fig. 21. Compressed air for the system (also used in surge tank and air inlet) is furnished by two 7 in. by 6 in. compressors, each with 15-hp motor and one 9 in. by 8 in. compressor with 25-hp motor, discharged into a storage tank at 100-lb pressure per sq in. The air discharges to the pipeline through a pressure-reduction valve that limits the pressure in the line to 5 to 10 lb per sq in. as desired.

It will be noted from Fig. 21 that the intake is not housed. Minimum temperatures at the site have been as low as 20 degrees below zero Fahrenheit. There has been no heating during the twenty years that the plant has been in service, and the bubbler system at trash racks, air inlet, and surge tank has kept the plant free from ice troubles.

An air bubbler system is used also at Grand Coulee [3] in front of trash racks and spillway gates to prevent ice troubles. The depth of water to the

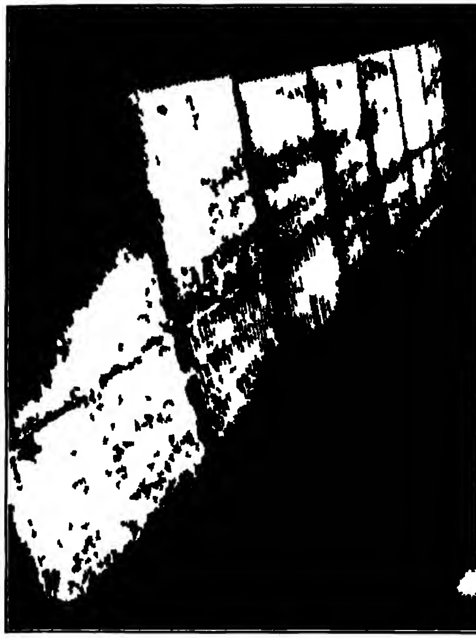


FIG. 19 Holtwood development top sections of racks raised after start of frazil ice run (Pennsylvania Water and Power Co.)

FIG. 20 Frazil ice on trash rack at Safe Harbor plant with rack removed from slot (Pennsylvania Water and Power Co.)

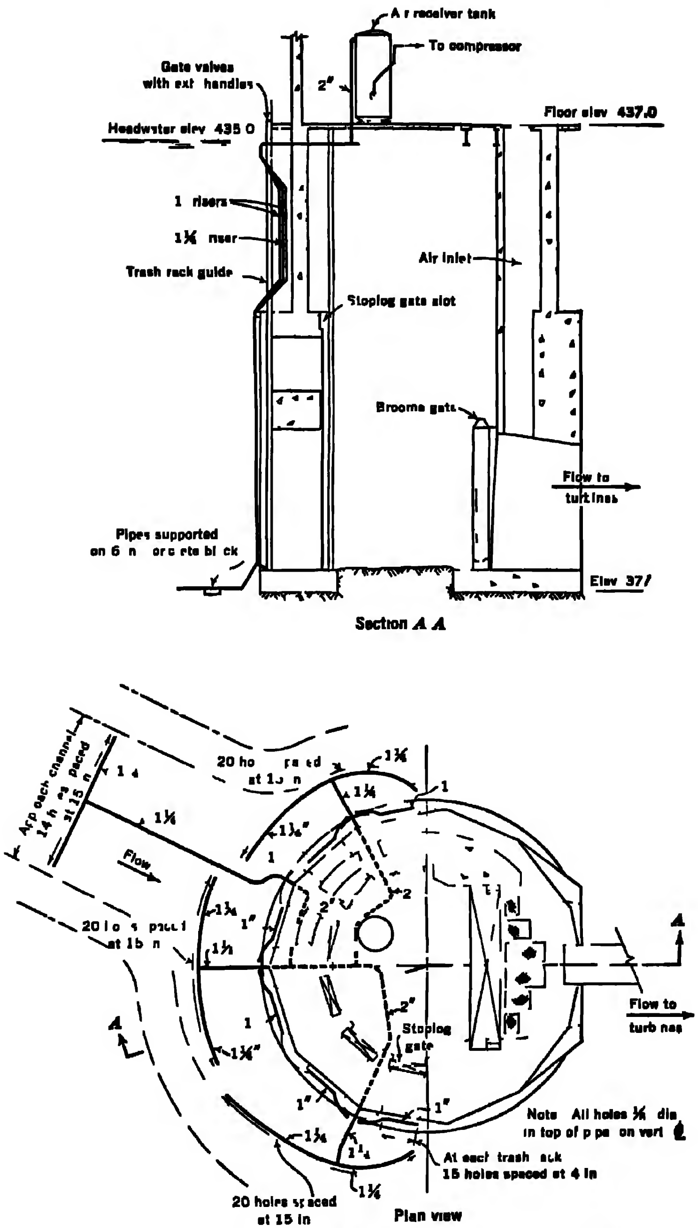


FIG 21 Sections of Rocky River intake showing arrangement of bubbler system (Connecticut Light and Power Co)

point at which compressed air is released varies from 10 to 36 ft. Nozzles are drilled holes ⅛ in. in diameter. There is a total of 1480 such nozzles (spillway spacing is 12.5 ft, trash-rack spacing 5 ft). The capacity of each of the ⅛-in. orifices was found to be 2 to 3 cu ft of free air per minute with a differential pressure across the nozzle of 2 to 4 lb per sq in. The air branch lines are ¾-in. and 1-in. copper pipe. The main air lines, 4000 ft long, are 4-in. galvanized-steel pipe, served by four electrically operated compressors, with a capacity of 380 cu ft of free air per minute each at 40 lb per sq in. They are located in the inspection galleries at convenient points.

(*h*) *Removal of the Racks.* At the Holtwood and Safe Harbor plants on the Susquehanna River, the Cedar Rapids plant on the St. Lawrence, and a number of other developments where there is no provision for preventing frazil and anchor ice from adhering to the racks, the upper sections of the racks are removed when ice starts to run. (See Figs. 19, 20, 15, and 18.) At the Cedar Rapids plant in Canada, the upper half of the racks in each bay is made in one section, to be readily removed. Usually little or no débris is present in the water when anchor and frazil ice is running, and the removal of the racks has caused no inconvenience or damage from that source. (See also Section 9.)

7. Velocities through Gates. The authors recommend that the velocities through gates in intakes be not greater than that given by the following formula, and that even for very high heads they be limited to 25 ft per second.

$$v = 0.12\sqrt{2gh}$$

in which v = average velocity through wide-open gate;
h = head from center line of gate to normal water surface.

Thus for a 20-ft head the above criterion requires that the velocity through the gate should not exceed 4.2 ft per second Similarly, for a 60-ft head, it should not exceed 7.8 ft per second; for a 150-ft head, 11.8 ft per second; and for a 400-ft head, 19.3 ft per second.

For successful operation, the velocity through the gates, for a given maximum discharge, may vary between wide limits. However, as the higher velocities result in lower gate cost but greater eddy loss and less power output, there is usually one size of gate that will make for the greatest economy of design. Figure 22 gives permissible velocities through intake gates based on the above formula.

Velocities through the gates at full load vary in different developments from 2 to 19 ft per second, depending on the head on the development, the shape of the load curve, and the design of the gate, among other factors. High-head developments permit high gate velocities, as the percentage of head loss is relatively small. It is evident that markets having a peak demand of very short duration will permit higher full-load velocities than those for which

the peak demand is of long duration. Common velocities are 2 to 4 ft per second in very low-head plants, 4 to 7 ft per second in medium-head plants, and 7 to 20 ft per second in very high-head plants.

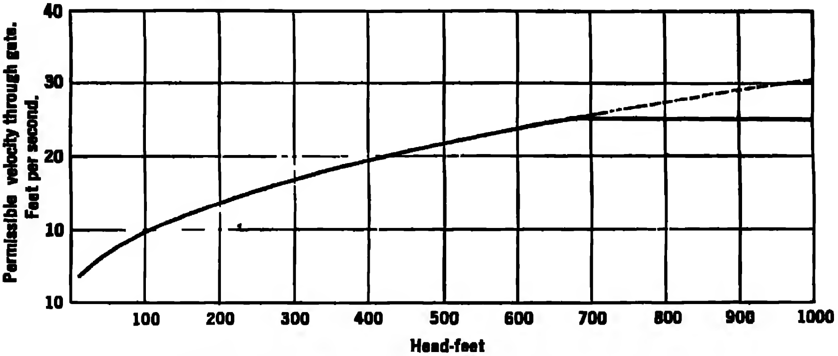


FIG. 22. Permissible velocity through intake gates.

8. Intake Structures. As explained in Section 1, intakes are divided into two main types, with no clearly defined line of demarkation. The essential requirements of the two types are the same, but the details and type of equipment may be radically different on account of the differences between the depths of water in which they must be operated.

When high-pressure intakes are used in conjunction with an earth or rock-fill dam, the structure usually takes the form of an isolated tower and the details of the structure are radically different from those of a low-pressure intake.

In low-pressure intakes, where there is considerable danger of complete and sudden plugging of the racks with ice or trash, it is customary to design all portions of the intake for complete stoppage of flow through the racks. In such cases the corresponding hydrostatic pressure on the racks, walls, and other portions of the structure is known so exactly and occurs so seldom that a relatively high unit stress may be used in the design. Reasonable working stresses are 25,000 lb per sq in. in the structural steel, and 33% more than the usual working stresses in reinforced concrete.

For low-head developments in warm climates, if the run of trash is very small, a maximum drop of water surface at the racks of 5 or 6 ft is a reasonable assumption, provided that the intake is not too isolated and that the installation is so important that only first-class, experienced operators are in charge. Under such circumstances, a greater drop at the racks would be the result of many weeks of neglect and would always be observed and corrected to prevent the corresponding loss of power.

As the depth of rack increases, the danger of complete stoppage of the flow becomes more remote. Thus, for high-pressure intakes at the outlet of a large reservoir, the amount of trash reaching the intake may be negli-

gible, and the water is drawn out of the reservoir so slowly that only the part of the racks near water surface could become plugged with ice. Therefore, only the lower portion of the structure need be designed for full stoppage when water surface is low, the reservoir small, and the danger from ice and trash correspondingly great.

The strength of the walls, buttresses, and other parts of the intake structure should be designed for all possible conditions of operation and unwatering for repairs. If the intake is divided into bays, each commanding a separate unit, as in Fig. 6, it is possible for only one unit to be running, the rack of that unit to become plugged, and the walls between that unit and the adjacent ones to be subject to full water pressure. Each unit should be investigated for stresses due to water pressure when either all or only a part of it is unwatered by the closure of gates or by the installation of stoplogs.

Further details of the two types of intakes are described in succeeding sections.

9. Low-pressure Intakes. (*a*) *General.* Details of low-pressure intakes vary greatly according to the requirements and the preference of the designer. Typical examples are shown in Figs. 15, 16, and 38 and Fig. 33 of Chapter 38.

The raking platform should be placed as close to water surface as possible: for this reason, it is frequently only slightly above ordinary high-water surface, with the main wall carried well above maximum flood elevation.

Low-pressure intakes frequently perform the functions of a dam and should be designed in accordance with the theory of the stability of dams. Full uplift must be considered on the top of the conduit below the gates, unless the top and bottom of the conduit are tied together with steel reinforcements, as they frequently are.

Usually a stoplog groove is located close to the upstream side of the gates to enable unwatering of the gates for inspection and repairs. The necessity for unwatering the rack supports is usually considered very remote, and only in isolated cases are stoplog grooves placed upstream from the racks. However, the walls between the bays are frequently allowed to project beyond the racks to provide a seat for an inclined bulkhead in case unwatering should become necessary.

(*b*) *Setting Gates.* If the seats for the gates are imbedded in the main portion of the concrete, it is very difficult to set them accurately. When accurate alinement of the seats is required, recesses are frequently left in the concrete, the seats are grouted in after the gate is set in position, and the gate and seats are held in contact to accurate position.

(*c*) *Air Inlets.* Air inlets are necessary in all intakes for closed conduits, as is more fully discussed in Section 31. These are provided by means of a vertical shaft in the concrete below the gates. Such shafts should be equipped with ladders to give access to the conduit at that point. Special requirements of air inlets at intakes, particularly in regard to the prevention of freezing, are explained in Section 31.

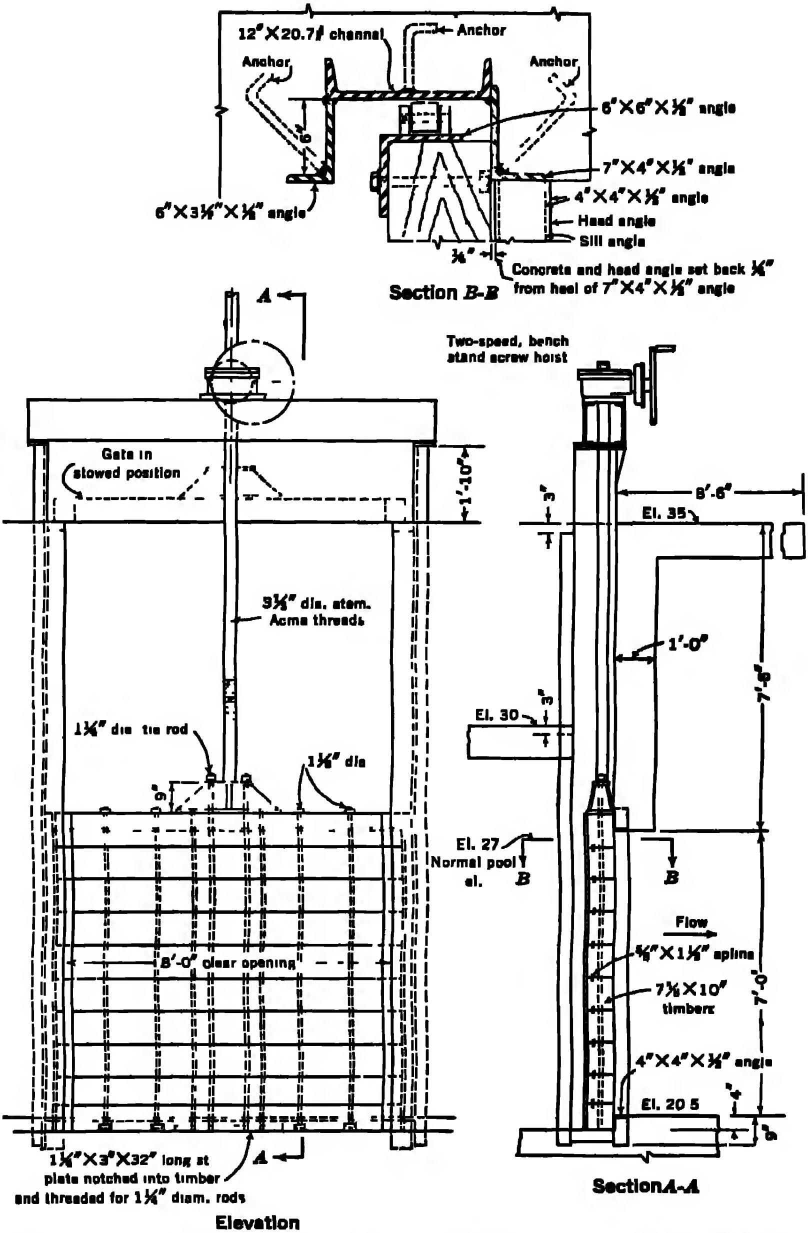


Fig. 23. Timber head gate. Union Lake Dam, Millville, N J. (Millville Manufacturing Co.)

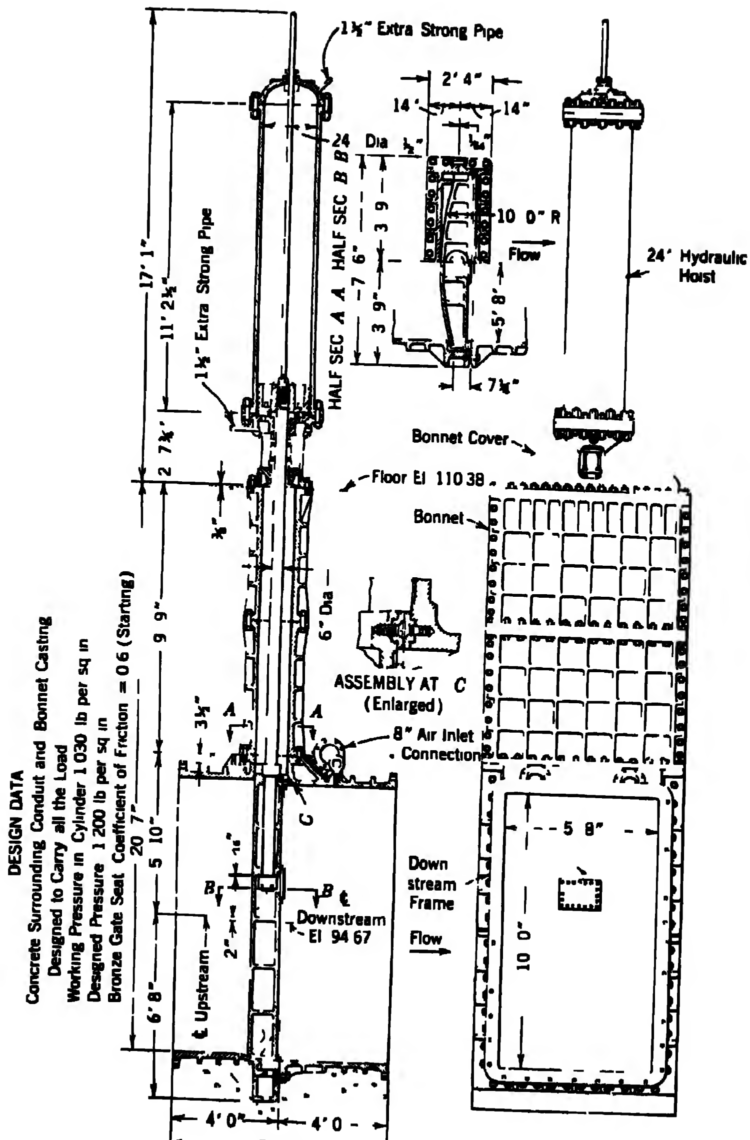


Fig. 24 U S Bureau of Reclamation slide gate

(*d*) *Housing.* The desirability of a superstructure or housing on the intake depends on climatic and other conditions and on the type of gate hoist provided. In very cold climates a superstructure to protect the rack tenders is advisable if much raking is necessary. The superstructure frequently extends over the racks, and a curtain wall extending below water surface is provided, as shown in Fig. 15. This arrangement seals the interior from the outside air.

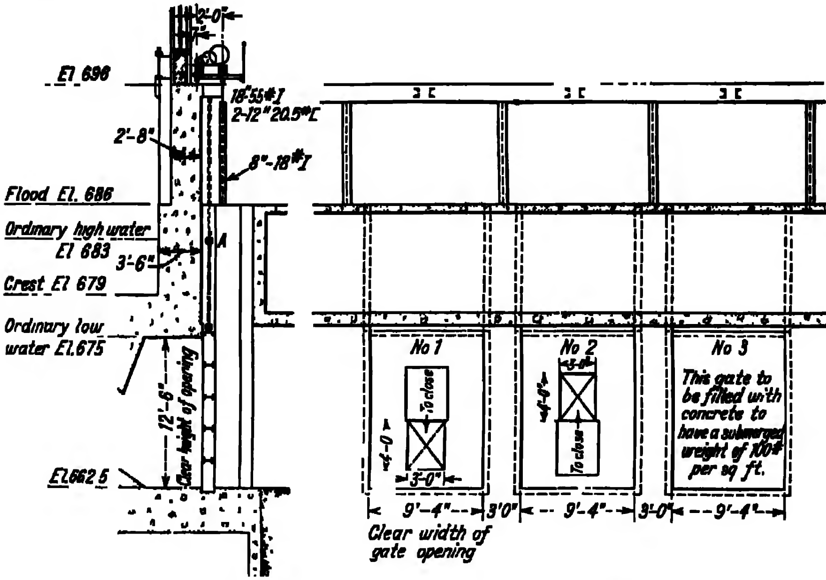


FIG. 25. Plain steel slide gate with filler gate.

If the racks are allowed to project above water surface and the air within the superstructure is kept warm, the transmission of heat down the rack bars may reduce the amount of anchor ice which will cling to the racks. A heated superstructure will prevent the formation of surface ice at the gates. (See Section 6.)

(*e*) *Intake Crane.* In some instances, where an intake superstructure has been provided, an ordinary powerhouse traveling crane has handled both the gates and the racks. In the absence of a superstructure, a gantry crane has been used. For large plants it is often considered necessary to have individual hoists for each gate.

(*f*) *Ice Sluice.* When much trash and cake ice must be taken care of, it is of considerable assistance to the rack tenders to provide a sluice over the trash racks to carry it away. Such an arrangement, shown in Figs. 16 and 17 and Fig 33 of Chapter 38, consists of a sluice, one side of which is a weir at the top of the racks The water can be allowed to pass over this and

carry with it the floating ice and trash, as well as the ice and trash removed from the racks. The sluice discharges below the dam. The elevation of the crest of the weir in some installations can be adjusted by means of stoplogs in order to regulate the depth of overflow.

During periods of abundant river discharge the water is allowed to pass over the weir continuously; but, as the capacity of the sluice is limited, the crest of the weir must be adjusted to the elevation of headwater surface in order to prevent choking of the sluice with water. For operation to be most effective, there must be a decided drop at the weir, and the discharge must be limited so that backwater in the sluice will not submerge the weir.

During periods of low river discharge, when there is little trash or ice to dispose of, the stoplogs on the weir are extended above headwater surface, thus stopping the flow over the weir. The trash is then raked over the stoplogs into the sluice and flushed out at intervals by the opening of a gate at the lower end of the sluice.

The sluice should be made as deep and wide as the type of intake will allow. Considerable width is required particularly if large blocks of cake ice are to be disposed of. The raking platform should be cantilevered out, as indicated in the illustrations, to avoid the necessity of other supports that would prevent the cake ice from passing into the sluice. The stoplogs at the Deferiet weir are held between removable I beams resting on the permanent weir crest and supported at the top against the raking platform.

(g) *Contents of Intakes.* The cost of the intake is frequently a very small portion of the total expense of the project, and so preliminary estimates are frequently made without the aid of a detailed design of the intake. For these circumstances Fig. 6 was prepared for this book by F. H. Burnette. It gives approximate quantities of materials in one type of intake for various discharge capacities and drawdown of pond. The designs upon which the quantities of Fig. 6 are based are as follows:

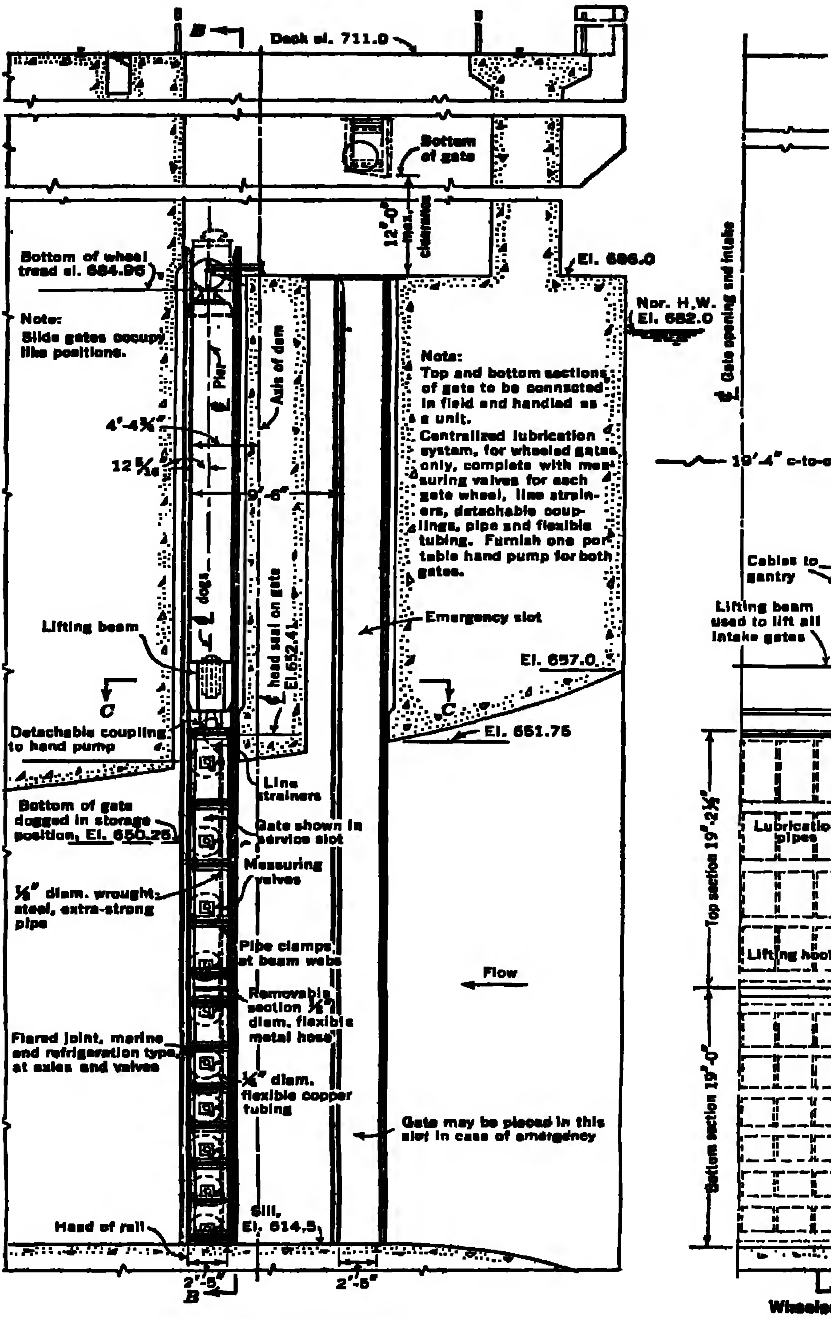
Elevation of top of intake	6 ft. above crest of dam
Velocity in pipe	8 ft per second
Velocity at racks (average drawdown)	1.75 ft per second *
Velocity at racks (maximum drawdown)	2.00 ft per second *

The quantities shown in Fig. 6 are for one unit of the intake. For N units, multiply the quantities by N and then add the following:

For concrete: 2 ft on each end to make end walls 4 ft thick.
For reinforcing steel: ¼ of that given for one unit.
For racks and supports: no addition.

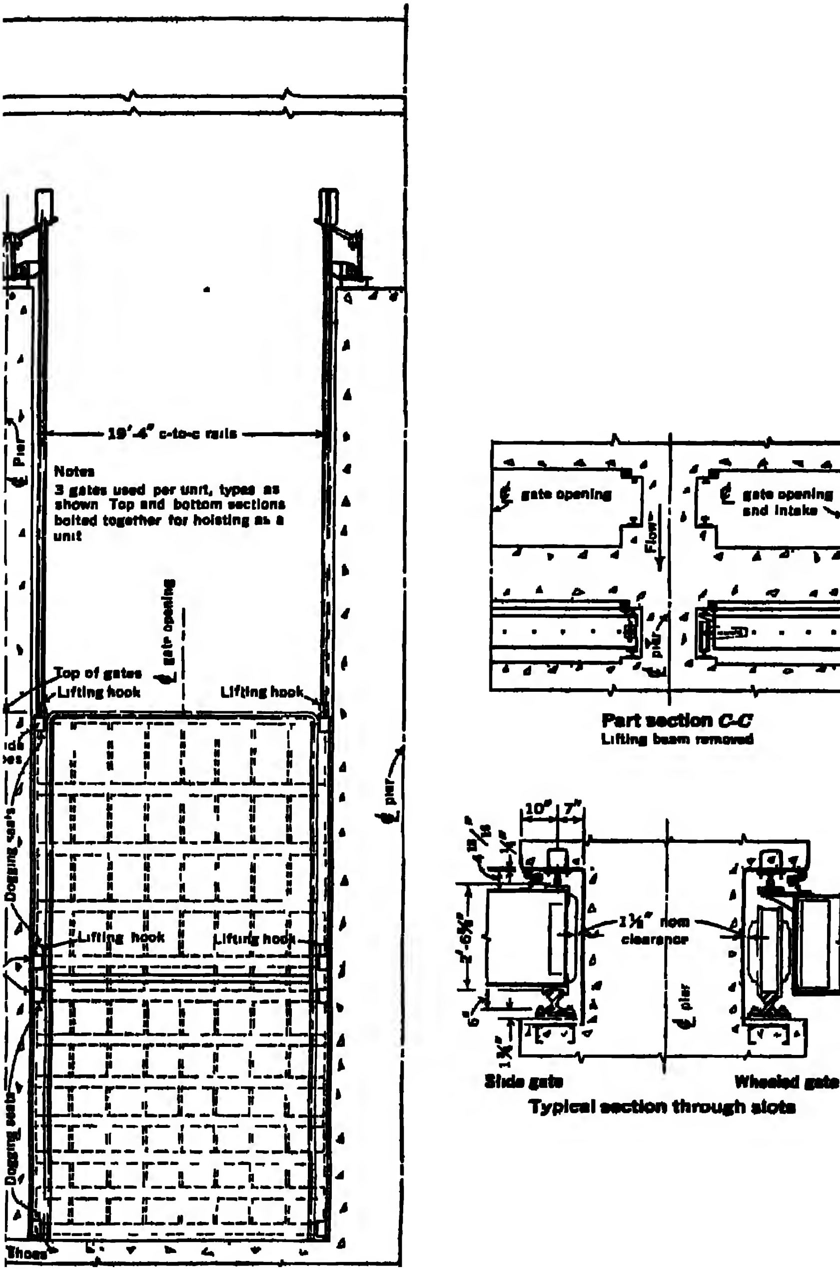
All concrete for necessary wing walls and for foundations below that indicated should also be added. Quantities should be altered for differences between assumptions and actual conditions, particular attention being paid to the elevation of the top of intake above the crest of dam, which may not correspond to actual floodwater elevation.

* Whichever requires the larger area of racks.



Sectional elevation A-A

FIG. 26. Chickamauga intake with wh



Slide gate

eam elevation *B-B*

l slide gates. (Tennessee Valley Authority.)

Quantities of steel in racks and supports are based on full stoppage of flow by rack plugging and a stress of 20,000 lb per sq in. under such conditions. The racks are 3 by ⅜ in. spaced 3 in. centers.

10. High-pressure Intakes. High-pressure intakes can be built in a variety of forms. If a dam is of concrete, the details of the intake differ very little from those of the low-pressure intake described in Section 9, except in the type of gate. If the dam is an earth or rock-fill structure, the intake usually takes the form of a tower located near the foot of the upstream slope, although special types of intakes have sometimes been used in shafts of outlet tunnels. Typical modern high-pressure tower intakes are shown in Figs. 40, 41, 42, and 43.

Tower intakes should be sufficiently stable to resist ice thrust, wind pressure when the pond is empty, and, in some locations, the shock of earthquakes.

It is unfortunate that no accurate data are available on the probable thrust of ice. A general discussion of ice thrust on dams is given in *Engineering for Dams;* * but it is very improbable that many existing tower intakes could successfully withstand ice thrust of the magnitude used in the design of some dams. It is therefore necessary to compare the proposed design with the dimensions of existing intakes of about the same height under similar ice conditions.

The Davis Bridge intake, shown in Fig. 42, has two gates in the bottom of the tower, inclosed in a cast-iron section of the conduit which excludes the water from the interior of the tower. Leakage, in towers of this type, should be drained to a point below the dam. There should always be at least two gates, and provision should be made so that one can be bulkheaded off for inspection or repairs while the flow for the turbines is being passed through the other. This feature was incorporated in the Davis Bridge intake.

A bridge is frequently installed between the dam and the tower to permit easy access and the replacement of apparatus. Housing is usually provided on top of the tower in cold climates. If the apparatus in the intake is heavy, supports for a crane or chain block are advisable for lifting the apparatus out of the interior of the tower.

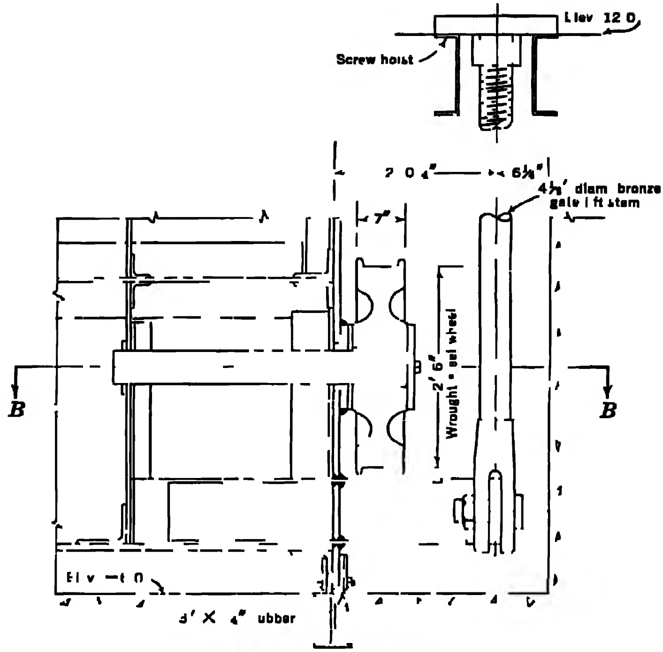
Much of the discussion of details of low-pressure intakes in the preceding section applies equally to high-pressure intake design.

11. Intake Gates. Data for estimating the weight of various types and sizes of gates will be found in Chapter 26. Intake gates and valves are readily divided into the following classes:

(*a*) Sliding gates Those that slide directly on their seats without the interposition of rollers. They are made of various materials but are all of the same general design (See Figs. 23, 24, 25, 26, and 32)

(*b*) Wheeled or tractor gates. The pressure is taken by wheels attached to the gate. The wheels may or may not be roller bearings. (See Figs. 26, 27, 28, and 33.)

* By William P. Creager, Joel D. Justin, and Julian Hinds, John Wiley & Sons, New York, 1945.



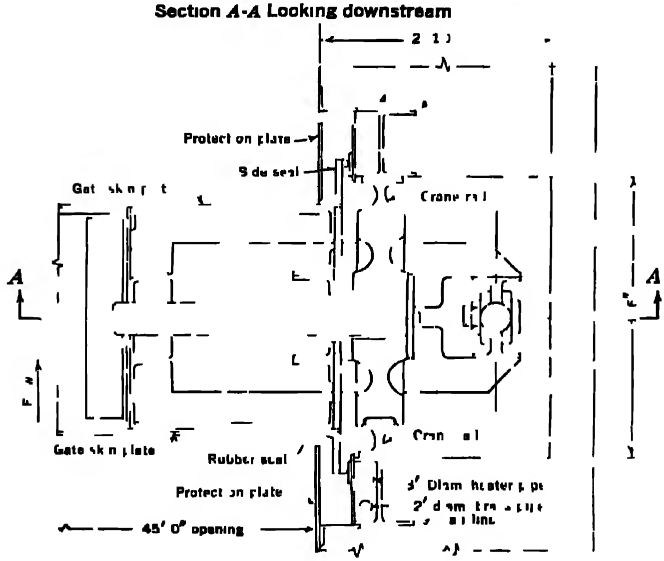


Fig 27 Section through wheeled gates Camden N J

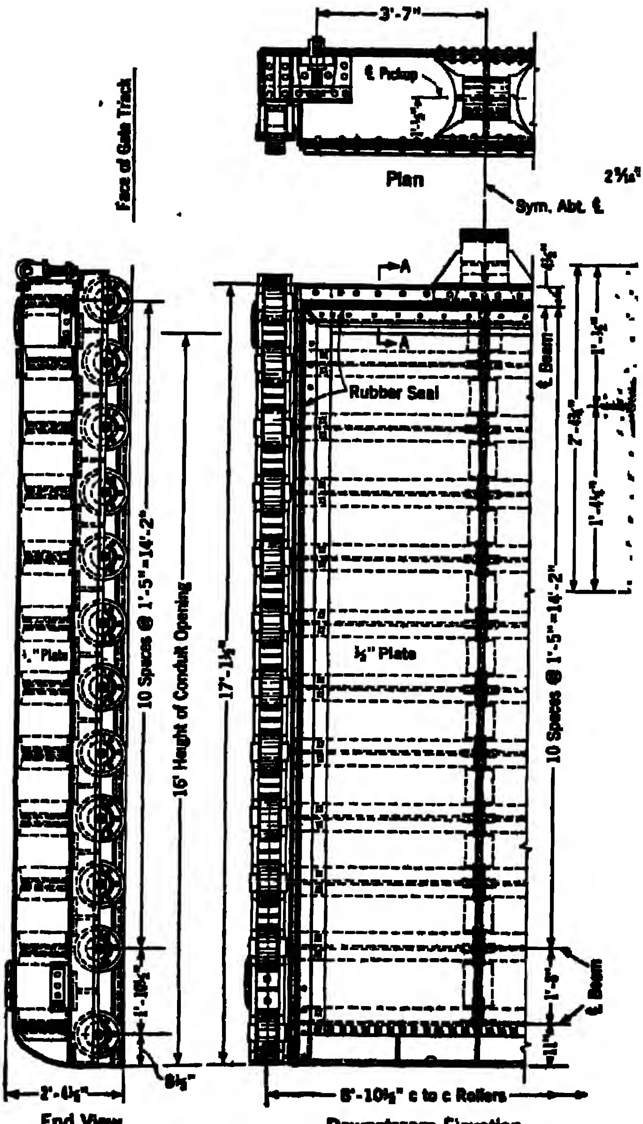
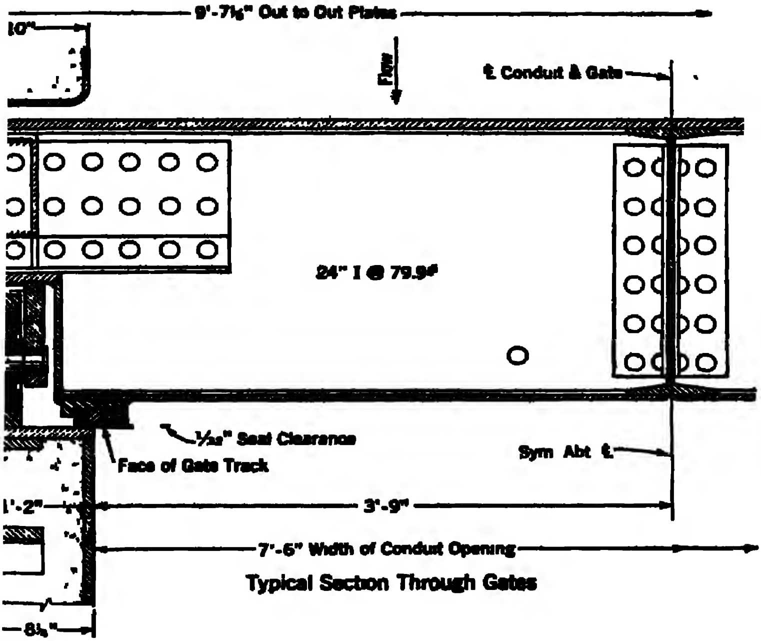
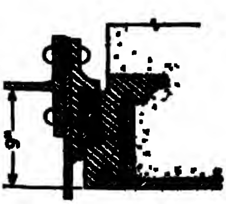


FIG. 28. Wheeled gate at Tionesta Dam



Typical Section Through Gates



Section A-A

ta, Pa. (U. S. Engineer Office, Pittsburgh, Pa.)

(*c*) Stoney gates. The pressure from the gate is taken by trains of rollers attached neither to the gate nor guides. (See Figs. 30 and 31.)

(*d*) Caterpillar gates. The pressure from the gates is taken by an endless chain of rollers. (See Figs. 29 and 43.)

(*e*) Taintor gates, which revolve about a spindle through the one extremity of the gate. Intake gates of this type are shown in Fig. 39.

(*f*) Butterfly valves. (See Figs. 34, 35, 36, 37, 38, and 42.)

(*g*) Cylinder gates (see Figs. 40, 41, and 43).

The type of gate to be adopted depends mainly on the size of the opening, the head on the gate, and the operating conditions, which will correspond to either of the following:

1. The gate must open and close against full operating head with free discharge through the intake.

2. The gate is required to operate only after the conduit is filled through a small auxiliary filler gate and the head on the gate reduced to a small part of the total head.

Gates controlling long pipelines are usually of the first operating condition so that they can be closed readily in the event of a break in the pipe or turbine casing. They are frequently equipped with motors operated from the powerhouse Gates for low-head plants and short pipelines are more frequently of the second, or filler-gate, class. A more detailed description of the design and operation of filler gates will be found in the next section.

For low-pressure intakes, plain slide gates, wheeled gates, caterpillar gates, stoney gates, and taintor gates are generally used. For high-pressure intakes, caterpillar gates, butterfly valves, and cylinder gates are generally employed.

Wheeled gates, stoney gates, caterpillar gates, and taintor gates serve also as spillway crest gates.

Whether one or more gates are used to control a single open or closed conduit is purely a matter of relative economy, and, in many instances, a number of different layouts must be investigated before the most economical arrangement can be decided on.

No standard set of rules can be given to fix the type of gate most adaptable to given conditions. In general, that type is adopted which, for desired operating conditions and grade of apparatus, can be installed for the least cost.

Butterfly valves or needle valves are frequently adopted for high-pressure duty; needle valves are preferable for extremely high heads.

12. Sliding Gates. (*a*) *General.* This term generally includes all vertical-lift gates. Sliding gates are usually built of wood, of structural steel, or, if small, of cast iron or cast steel. Small gates are frequently bronze-mounted, although there is little necessity for their being so unless they are to be kept closed for long periods. The larger steel gates are not usually bronze-mounted, although these and even some wooden gates have bronze strips to reduce the friction.

Sliding gates, in common with all types that have a continuous bearing under compression all around the opening, provide the minimum of leakage.

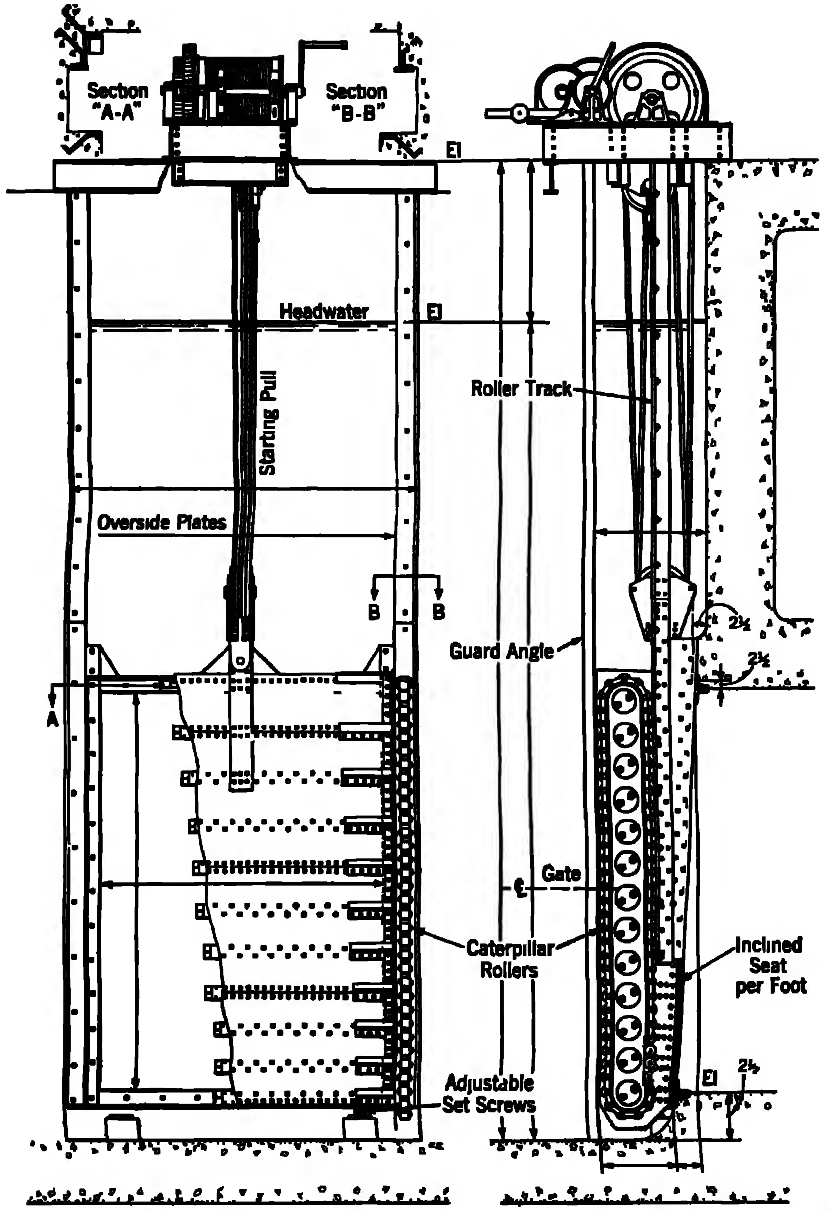


FIG 29 Caterpillar gate (broome self-closing sluice gate) (Philips & Davies, Inc Kenton Ohio)

Sliding gates have been used extensively for various heads, up to a hoist capacity of about 35 tons. They are usually the most economical type for low pressures and moderate sizes. However, they require a greater force to operate them than any other type. Consequently, for large sizes and

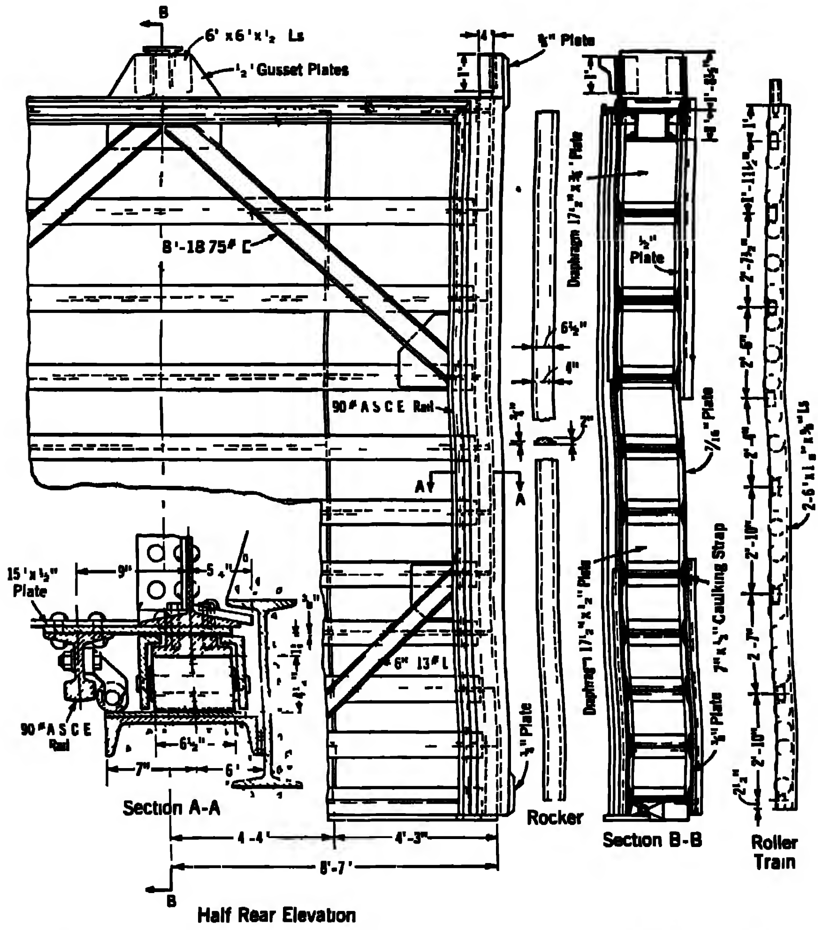


FIG 30 Stoney gate, Yadkin Falls, N C, development, Tallassee Power Co

heavy pressures, the capacity and cost of the hoist to operate them become very great. For this reason, economy frequently demands a more expensive but easier-lifting type of gate, or a small auxiliary filler gate through which to equalize the pressure while the main gate is in service. See Figs 23, 24, 25, 26, and 32.

(*b*) *Filler Gates* The filler gate is usually a sliding gate, sometimes mounted on the main gate itself but more frequently on the intake structure

controlling a passageway that connects with the conduit. Filler gates are used to fill the conduit behind the main gate, and thus they tend to equalize the water pressure behind and in front of the main gate so that the capacity of the hoist for the main gate can be minimized. (See Fig. 25.)

At the Conowingo plant, the intake gates are 27-ft-diameter butterfly valves (see Fig. 36, and Fig. 11, Chapter 30), located at the entrance to the

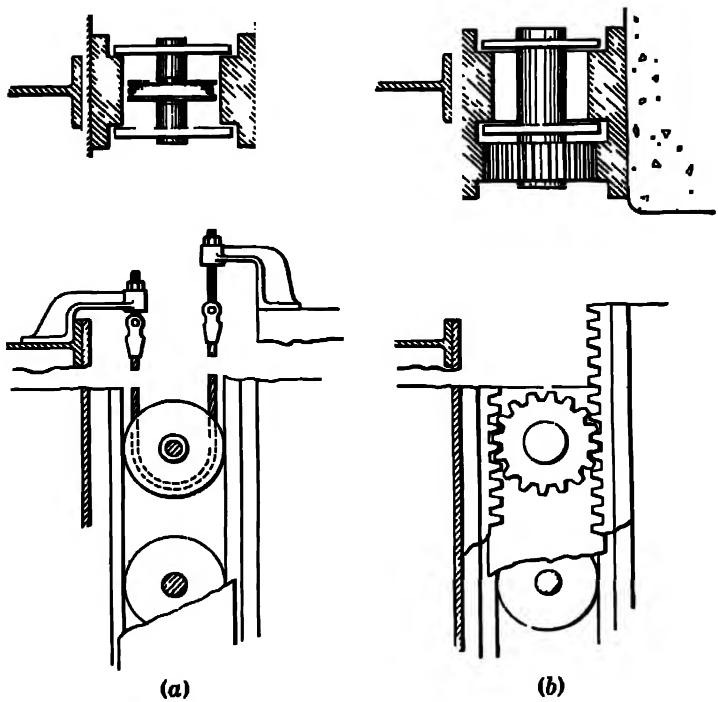


Fig. 31. Means of regulating travel of stoney-gate roller trains.

scroll case. Steel stoplog gates which constitute multisection plain slide gates are located in front of the butterflies. If the stoplogs are in place, the scroll case dry, and it is desired to put the unit in service, the butterfly valve is first opened, then the top 3- or 4-ft section of steel stoplogs is removed, and headwater is allowed to spill over until the scroll case is full and the pressure equalized. The remaining sections of the stoplogs or plain slide gate are then readily removed. Thus this arrangement takes the place of a "filler gate."

It is frequently assumed that the filler gate must be designed to permit the closing of the main gate under a leakage equivalent to the flow through two of the turbine gates. Thus, if the turbine has 20 wicket gates and the discharge of the turbine at maximum operating head is 1000 sec-ft, then the leakage under which the main gate must be able to close should be 100 sec-ft and the capacity of the filler gate should exceed 100 sec-ft at this head. A

smaller leakage is usually assumed for the opening condition, as, before the intake gates are opened, the turbine gates are accessible for temporary or

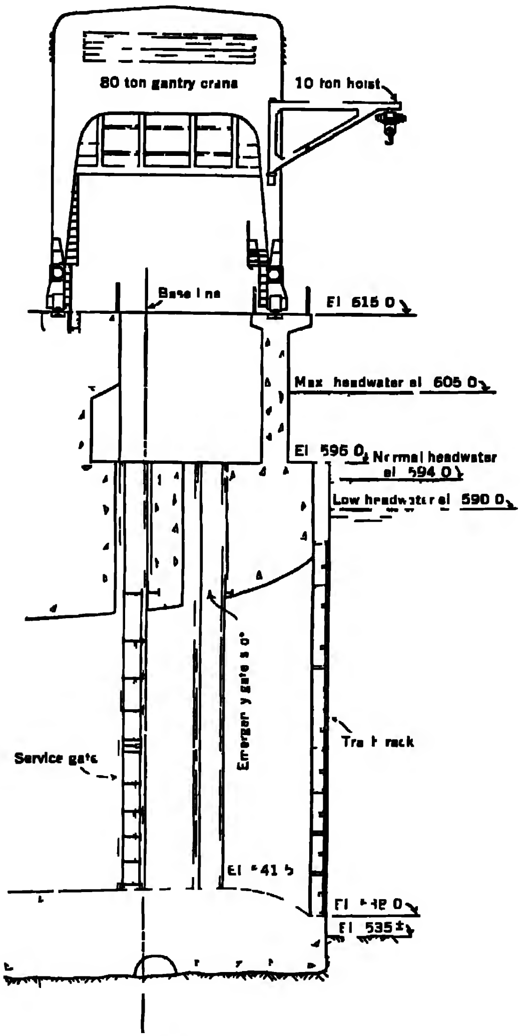


FIG. 32 Arrangement of intake gates, racks and crane Guntersville development (Tennessee Valley Authority.)

permanent repairs. In some cases, therefore, the main gate has been heavily weighted with concrete on the theory that the weight will assist closure and, with less leakage, the head against which the gate must open will be less than that for closure.

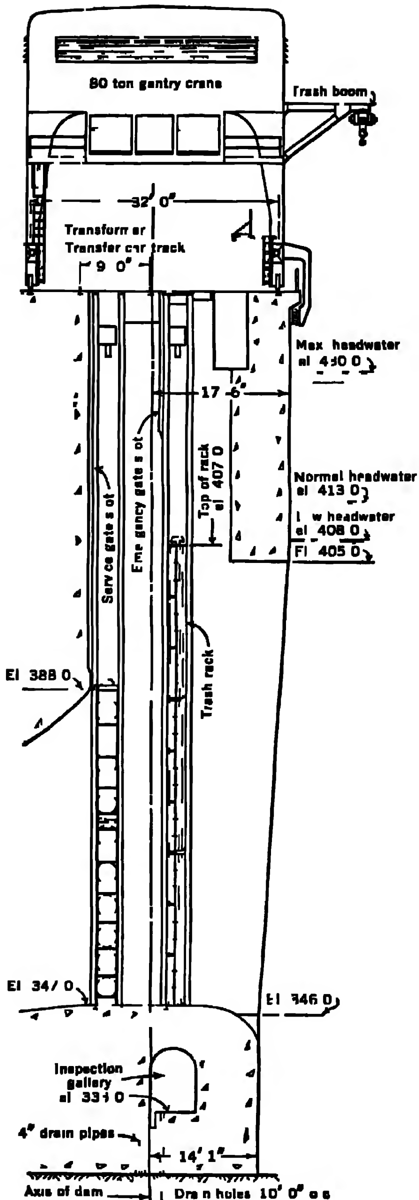


Fig 33 Arrangement of intake gates, racks and crane Pickwick Landing development (Tennessee Valley Authority)

There are two objections to filler gates:

1. The main gate cannot be closed if the leakage is greater than anticipated, and it is then necessary to place stoplogs or some other emergency means of stopping the flow, at a considerable expense and loss of time.

2. If the conduit is long, the filler gate, being small, will not fill the conduit quickly.

For these reasons, filler gates are seldom used in modern developments having long conduits. They are frequently employed, however, in very low-head plants with short conduits, particularly if they control the opening to a concrete spiral case turbine, where the danger of excessive leakage, except through the turbine, is practically nonexistent.

(c) *Timber Sliding Gates.* Timber gates are considered less permanent than steel gates, particularly if they are only partly submerged in the open position. However, as they are very easily replaced and the first cost is much less for gates of moderate size under low heads, they are frequently used.

In the East the most popular wood for timber gates is longleaf yellow pine. The western states furnish Douglas fir and California redwood. Shortleaf yellow pine, white pine, and white oak have also been used extensively.

The thickness of gate timbers can be proportioned according to the usual rules of timber design. The bearing pressure must be limited to that which will not cause brooming under friction. In general it should be limited to 100 lb per sq in. for softwoods and to 300 lb per sq in. for hardwoods such as white oak.

Figure 23 shows details of a typical wooden gate with steel screw stem. Wooden gates are often built with timber stems They frequently are provided with splines for tightness and have stiffening planks bolted or spiked to them. The stems are always bolted to the gates. To prevent the upper and lower timbers from loosening, it is customary to bolt together the first few timbers with a vertical throughbolt, and sometimes such bolts are continuous from the top to the bottom of the gate, as in Fig. 23.

13. Wheeled Gates. Gates of this type (shown in Figs. 26, 27, 28 and 33) are variously called tractor gates, roller-bearing gates, and trunnion gates. Their distinguishing characteristic is that the water pressure against the gate is carried on wheels attached to the gate. It is an excellent form of gate, but the sealing problem is usually more difficult than with some other forms such as caterpillar gates, butterfly valves, and cylinder gates.

14. Caterpillar Gates. These are among the most satisfactory forms of head gates. With the gates in adjustment and closed, the leakage may be made insignificant. They are, therefore, in demand for plants in which the leakage of water with the unit closed down is a material item of cost.

A caterpillar gate or broome gate, as it is sometimes called after the engineer who invented it, as shown in Fig. 29. This type is used for both high-pressure and low-pressure intakes. It has been employed under heads as high as 200 ft and on many hydroelectric and flood control developments.

Each side of the gate is equipped with a continuous chain of "caterpillar rollers" which runs on a vertical seat in a slot in the intake piers and travels around the gate as it is raised or lowered. The chain eliminates friction to a very large extent and allows the gate to lower solely by its own weight under free discharge of the opening. Therefore, the drum type of hoist, described in Section 26, is generally used for the operation of caterpillar gates.

The chain consists of a series of steel rollers about 4 in. in diameter and spaced about 5 in. center to center. The links of the chain are of steel and are separated from each other and from the steel rollers by bronze washers. The link pins are of bronze, so that there is no frictional contact of steel on steel, which might corrode and interfere with proper operation. The chain runs on a cast-iron track fastened to the gate.

Besides the method of movement, another feature of the caterpillar gate is the inclined seat. The motion of the gate is vertical. The seating surface of the gate on the sides is inclined at about ½ in. horizontal to 1 ft vertical. When the inclined seating surface at the sides of the gate comes in contact with the corresponding inclined seating surface of the frame, the gate is closed and there is no further movement, as no sliding on the seat of the frame occurs in a properly adjusted gate. A very nice adjustment is required in order to eliminate any sliding or wedging when these two machined seating surfaces come together. This is accomplished by two large adjusting screws set in the lower I beam. Once the adjustment is properly made, there should be no trouble from wedging of the seats. There is very little leakage through this type of gate. The gates are manufactured under the Broome patents by Philips and Davies of Kenton, Ohio.

For the force required to operate caterpillar gates, see Section 26.

15. Stoney Gates. Stoney gates have been frequently and successfully used for head gates in low-pressure intakes as well as for flood gates. They are structural-steel lift gates that bear on trains of rollers working in the guides. The roller trains, which are not connected to the gate at all, consist merely of a large number of hard-steel rollers held together in a light steel framework. As the gate is raised it actually rolls on the rollers (if pressure is back of the gate). When the gate is coming up, the train of rollers moves up also, but only half as fast as the gate. A typical example of the stoney gate is found in the Yadkin Falls development (see Fig. 30).

The stoney gate is capable of lowering solely by its own weight under free discharge of the opening. Therefore, the drum type of hoist generally is used for its operation. At Yadkin Falls, however, the screw stem type of hoist was adopted.

A satisfactory way to secure the roller chain is to attach a wire rope to the underside of the operating bridge and pass it under a sheave on the roller train, then up to a bracket on the top of the gate itself. Figure 31 makes clear the means of regulating travel for the stoney-gate roller trains. Methods of sealing stoney gates are discussed in Section 19.

16. Taintor Gates. Sector gates, usually referred to as taintor gates after Captain Taintor, the inventor, are often adaptable as head gates and have frequently served as such. Taintor head gates are shown in Fig. 39.

They are usually made of steel throughout or steel with wooden facing. This type of gate is most suitable where the fluctuations of water surface are so limited that the top of the gate is at all times at or above water surface. They have been used, however, where water surface is above the top of the gate.

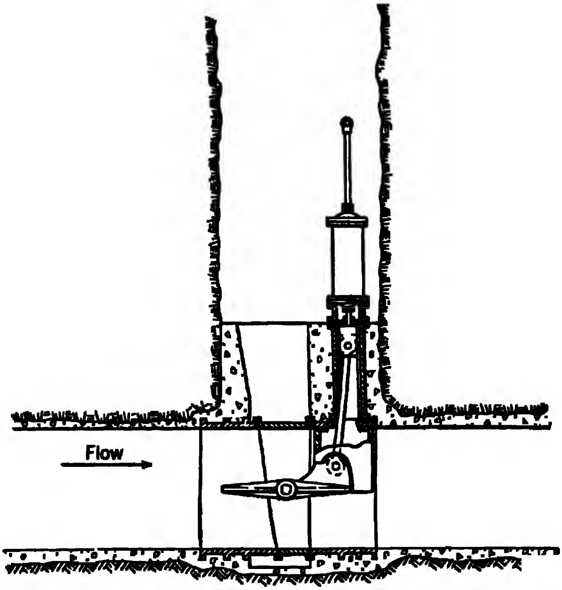


FIG 34 Dow valve (S. Morgan Smith Co., York Pa., and Coffin Valve Co., Neponset, Mass.)

Taintor gates, when submerged, are hard to keep watertight, both at the top of the submerged gate and at the lower corners. The only objections to them are this difficulty and the space they occupy horizontally. But, with proper attention to design, installation, and maintenance of seals, they can be kept just as tight as any other type of gate.

The gates are true sectors of a circle, and the water pressure, therefore, passes through the pivot. Consequently, they are capable of closing under free discharge by their own weight, and the drum type of hoist is used to operate them.

In the taintor gate, the damming surface is a section of a cylinder that is arranged to rotate about a fixed horizontal axis and is generally concentric with it. Its merit lies in its mechanical simplicity. The load is carried on two bearings, one at each end of the gate on the axis. The bearings are mounted on piers located at approximate intervals on the crest.

Taintor gates are usually raised by means of cables or chains acting simultaneously at both ends. Since the angular travel in the bearings is small

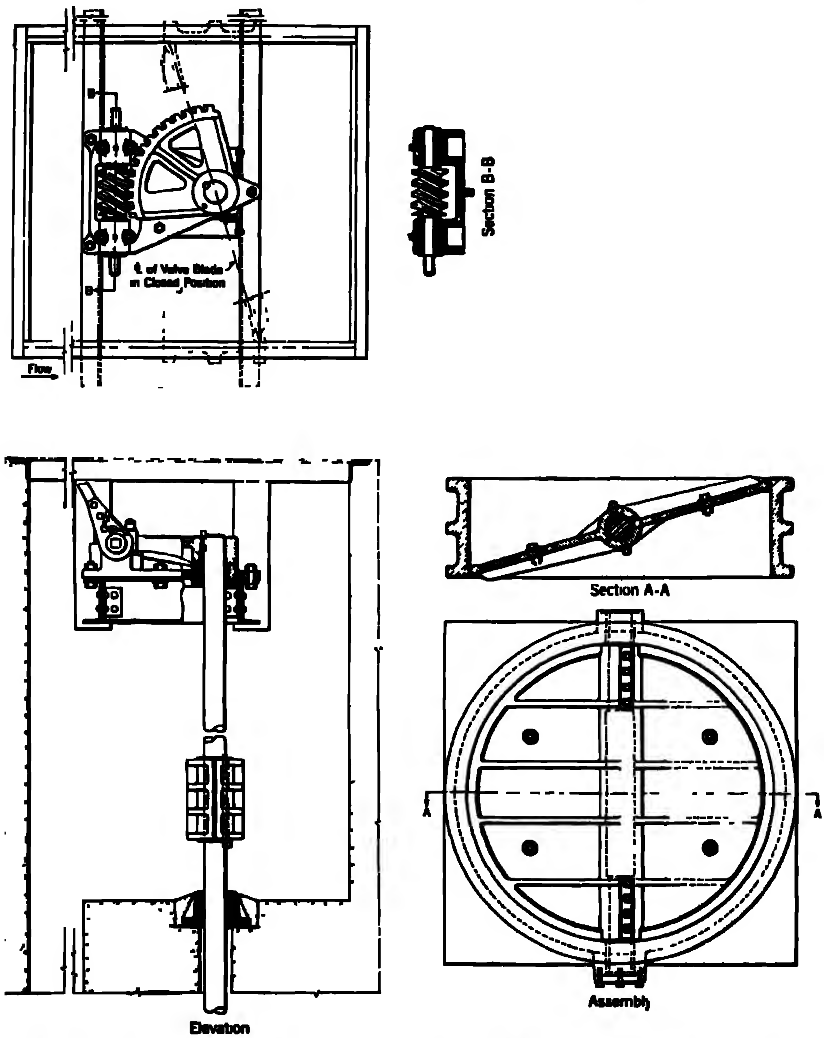


Fig. 35. Butterfly valve, Tygart Dam, Grafton, W. Va. (U. S. Engineer Office, Pittsburgh, Pa.)

when the gate is raised from the closed to opened position, the work done in overcoming the frictional resistance is small.

17. Butterfly Valves. A butterfly valve may be used as a head gate, as in Figs. 36 and 38, or it may be used in a conduit on penstock near the

entrance to the scroll case. Examples of butterfly valves are shown in Figs. 34, 35, 36, 37, 38, and 42.

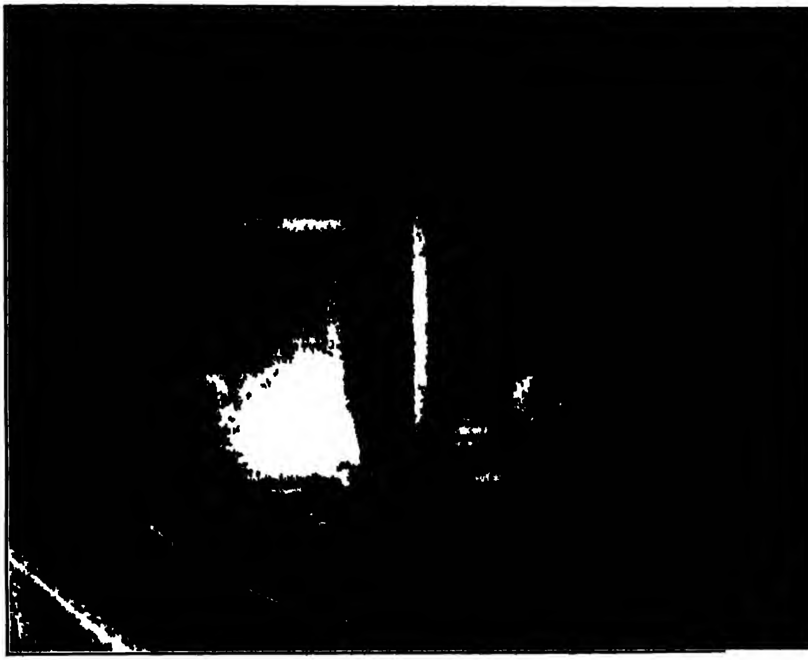


Fig. 36. 27-ft diameter butterfly valve at head of scroll case Conowingo hydroelectric development, Maryland. (Philadelphia Electric Co.)

Where prompt availability is required when the unit is shut down and where leakage of water during unit shutdown has a high value, the installation of butterfly valves near the entrance to scroll cases is economically

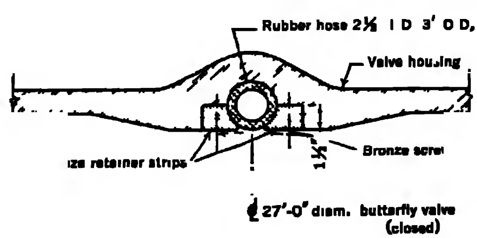


Fig. 37. Section showing rubber hose seal on 27-ft butterfly valve, Conowingo hydroelectric development.

justifiable. Butterfly valves can be made tight. On the other hand, if one is compelled to depend on wicket gates for units that may be required to go into service at any time, the leakage, even with good maintenance, may be 5% of the peak-load discharge of the unit.

Butterfly valves are adapted for use under both high and low heads. Small butterfly valves are used only under quite high heads; but structural-steel butterfly valves as large as 27 ft in diameter have been developed for use under all heads to 100 ft.

At the Conowingo plant, Maryland, the 27-ft-diameter butterfly valves (Figs. 36, 37, and Fig. 11, Chapter 36) normally act as head gates at the entrance to the steel scroll of each of the seven 54,000-hp turbines. These

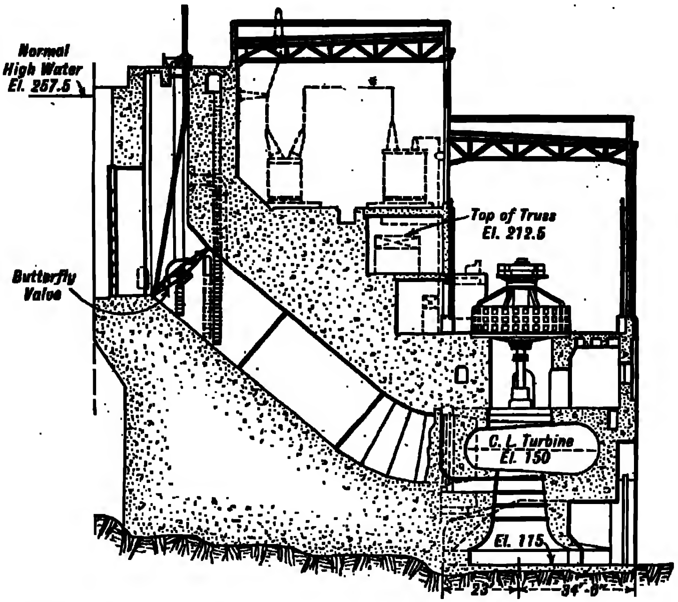


FIG. 38. A 16 x 22 ft butterfly valve, Saguenay River development, Quebec Development Co., Ltd. (*Eng. News-Rec.*, Vol. 93, p. 704.)

were the largest existing butterfly gates in 1947. There are also sectionalized steel stoplogs or plain sliding gates ahead of the butterfly gates, but leakage is relatively large for such head gates, and the speed of operation with a traveling gantry crane is slow. In fact, it may require 45 minutes or more to raise the two head gates of a single unit. On the other hand, the butterfly gates can be opened in 5 minutes under full head with the scroll case empty and the unit in service. With scroll case full of water and unit at rest, the unit can be placed in service in 1 minute or less.

Leakage through the valve, when closed, is controlled by a rubber seal around the periphery of the valve leaf, a distance of approximately 85 ft. The seal consists of a 3-in. rubber tube set in the valve housing, as shown in Fig. 37. When the valve is in the closed position, this tube is inflated with water under 70-lb pressure and caused to press against the brass strip around

the periphery of the valve leaf. Tests of this method of sealing show that the leakage under normal head conditions is about 0.5 cu ft per second.

The valves are of the vertical pivot shaft type, and the entire weight of approximately 126 tons, consisting of wicket, shaft, and operating lever, is carried on a roller-type thrust bearing located below the valve. The operating mechanism consists of an oil cylinder, located above the valve, with necessary levers. Interlocks are provided to prevent the inflation of the sealing hose except when the valve is in the closed position, together with devices for locking the valves when either open or closed. The butterfly-valve bearing is lubricated by forced grease, and it is necessary to have the valve in motion during lubrication. A control stand for each unit contains the various pressure and indicating dials and instruments as well as the control interlocks and levers for operation of the valve. This stand is located on an elevation 46 ft, on the same level as the governors. (See Fig. 11, Chapter 36.) The installation has given satisfaction since 1928, and only one hose seal out of seven has been renewed in this period.

An interesting design of a butterfly valve is the disk-arm type (Dow), shown in Fig. 34, in which the operating lever is attached directly to the lower half of the leaf so that the closing force is exerted right at the point where there is the tendency for the greatest deflection.

Butterfly valves can be operated either by a hydraulic cylinder or by a system of reduction gearing driven by a hand wheel or motor or a small water turbine. The hand- and motor-operated mechanism is used for most medium- and low-head installations; under relatively higher head, where the pressure is sufficient and the cylinder area can be reduced to an economical size, the hydraulically operated mechanism has many advantages, as water pressure is always available in a powerhouse while electric current is not, and as some 5 to 10 minutes may be needed to close a large valve by hand if the electric current fails. Hydraulic-operated cylinders should be bronze-lined to prevent rusting, and the cylinder area should be sufficient to meet all conditions. Oil pressure in the cylinder is preferable. The valve may tend to stick in the closed position if it has been wedged tightly shut and held closed for a long time. A cylinder should be designed to meet this condition. The motor-operated mechanism is readily adapted to remote control; that is, the control switches for opening and closing the valve can be adjacent to the valve, at the switchboard, or at any distant point, but limit switches should be carefully adjusted to stop the motor when the valve reaches the full open or closed position. Care should be taken to prevent the valve from jamming when it reaches the closed position. Frequently, indicating lights are provided to inform the operator at the switchboard of the position of the valve. Two lights should be sufficient, one for open and one for closed position, as there should be no need to use a valve in the partly open position.

Usually the pivot shaft is extended on both ends of the housing so that there is no end thrust on the shaft, and a readily adjustable stuffing box

should be provided at each end The trunnions should have renewable bushings, and some provision should be made for forcing grease into these bearings By-pass valves are usually provided on the larger sizes, but they are

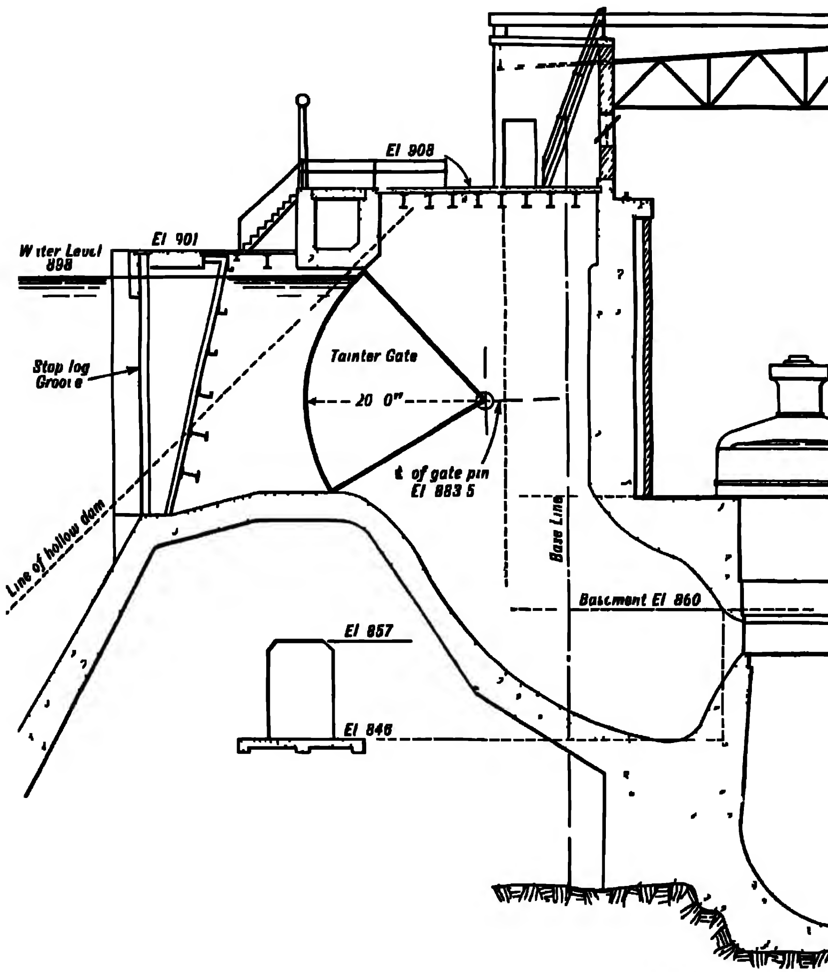


Fig 39 Tainter intake gate at Wissota development Wisconsin

not necessary for medium heads as it is an easy matter to open these valves a slight amount Plants with long penstocks usually have some form of valve at the lower end of the penstock where it connects with the turbine casing, and frequently another valve is located at the upper end of the pipeline These upper valves sometimes take the place of the head gates

18. Cylinder Gates. A cylinder gate consists of a steel cylinder open at the top and bottom and having balanced water pressure on the inside or outside surfaces. They are made in a variety of forms, of which those in Fig. 40 are examples.

Sketch *A* shows a cylinder gate with water pressure on the outside. The gate can be lifted by cable or light screw-stem hoist, as the water pressure is completely balanced and the only force to be overcome is the weight of the gate. The bottom of the cylinder rests on the seat *X* and when the gate

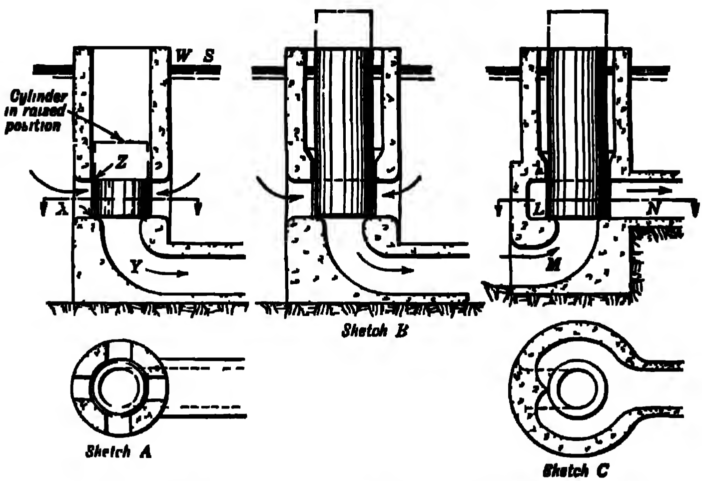


FIG. 40. Diagrammatic sketches of cylinder-gate intakes.

is lifted the water passes through conduit *Y*. The sill at *X* is usually a heavy iron casting provided with seals. Gates of this type are shown in Fig. 41.

Difficulty is usually experienced in making this type of cylinder gate tight at *Z*, and for this reason the cylinder is sometimes extended to water surface, as indicated in Sketch *B*.

In Sketch *C* the water pressure is on the inside of the cylinder, thereby eliminating the necessity of inside stiffeners. The water passes through the passages *M* and *N* by way of the chamber *L*.

19. Gate Seals. Figure 46 shows the most common types of gate seals. Direct metallic contacts like those in Fig. 46*a* require accurate workmanship and may suffer from abrasion where the water carries sand or silt. The poured babbitt metal seal shown in Fig. 46*b* presents a convenient method of providing a close-fitting metallic contact. Wood contact surfaces, as shown in Fig. 46*c* and 46*d*, give good service if they are not allowed to dry out. Flexible rubber sealing devices, as illustrated in Figs. 46*e*, *f*, and *g*, are sometimes added to the bottom of contacts of gates to secure greater water-tightness. A rubber hose seal that has given satisfaction for a butterfly gate

is discussed in Section 17 and shown in Fig 37. Side seals are usually of the flexible type. Figs. 46*h*, *i*, *j*, *m*, and *n* show typical rubber seals. The flexible plates with wood contacts, like those in Fig 46*k* are used where

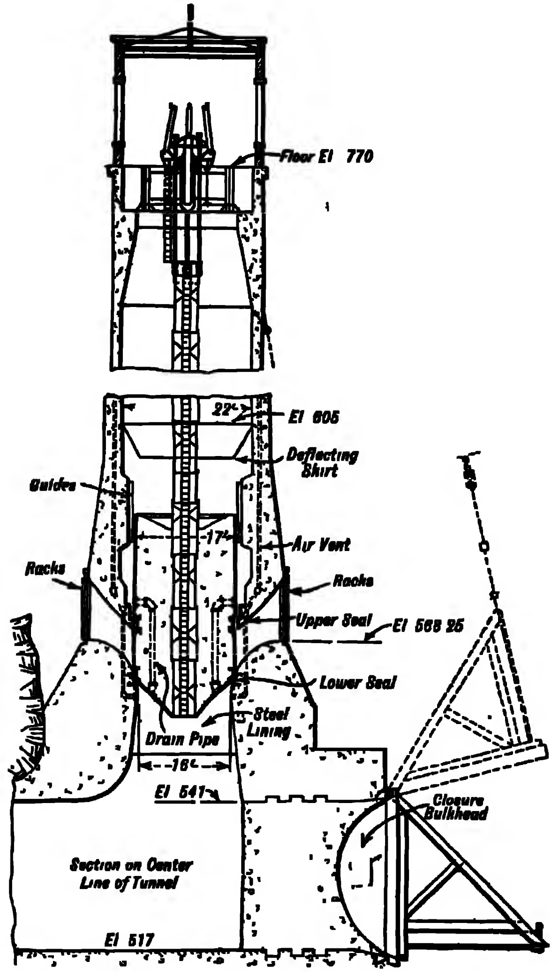


Fig 41. Sixteen-foot cylinder gate for Dix River Dam. (*Eng. News-Rec.*, Vol 94. p. 1059.)

operating conditions are severe. Flexible bronze seals in Fig. 46*l*, are also satisfactory.

Steel pipe, wooden needles, rubber hose, and Manila rope are sometimes used as temporary or occasional seals. Rubber belting makes a very respectable side seal for taintor and stoney gates (see Fig. 46h).

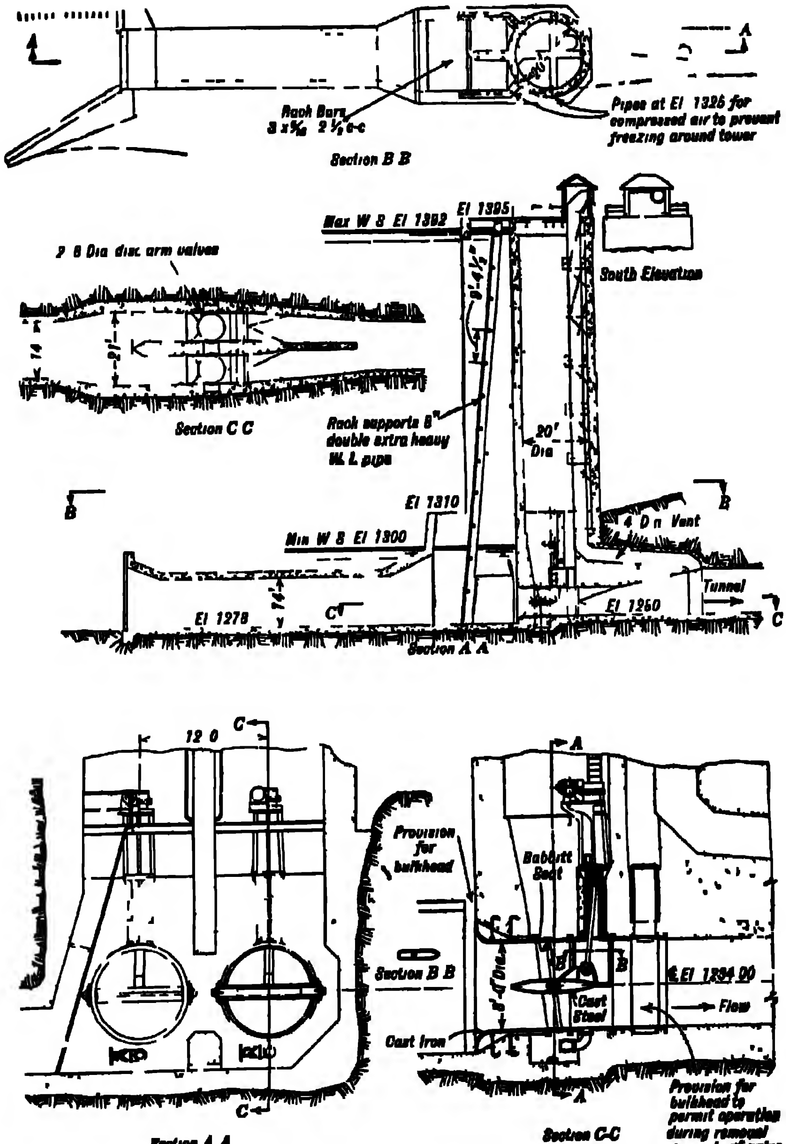


FIG 42 High-pressure tunnel intake Davis Bridge development, Vermont (New England Power Co)

The Yadkin Falls gates were made tight on the sides, as shown in Fig. 44, by means of rubber belting fastened on the sides of the gate and turned through an angle of 90 degrees so that it rubs along the face of the intake pier just upstream from the guides or, better yet, along the face of a channel

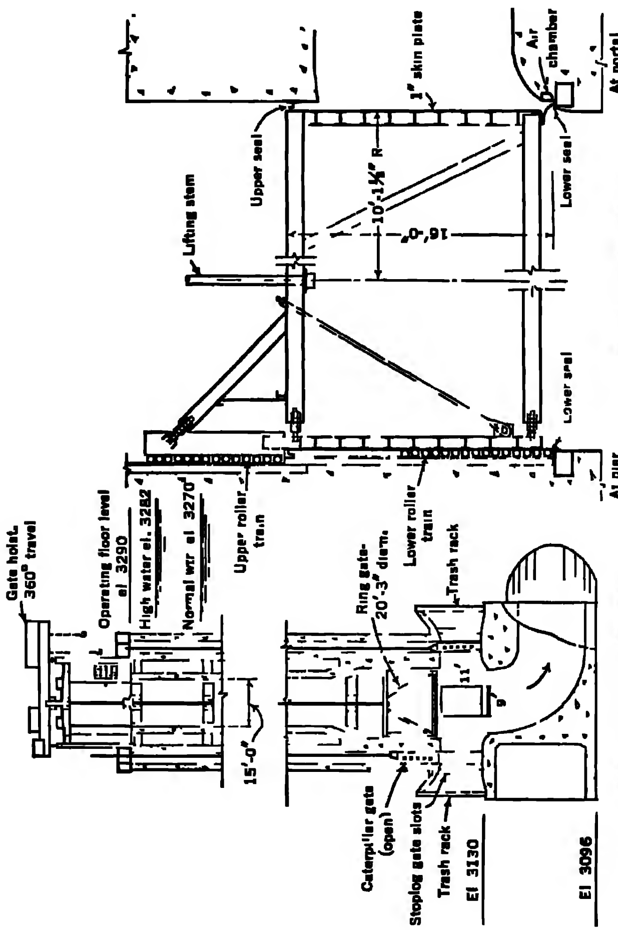


FIG 43. High-pressure intake with cylinder gate and caterpillar guard gates. Kingsley Dam Nebraska.

or angle set in the concrete, as in Fig. 46*h*. Five-ply best-quality rubber belting should be used for this purpose. The belt should be held to the face of the gate by a ⅜-in by 2- or 3-in strip of flat steel, which is fastened to the face with the belt, between it and the face of the gate, by means of about ⅝-in bolts spaced about 6 in. center to center.

The bottom of the gate is often made watertight by scribing to the sill a timber set in the bottom of the gate as in Fig. 46*d* A more effective means

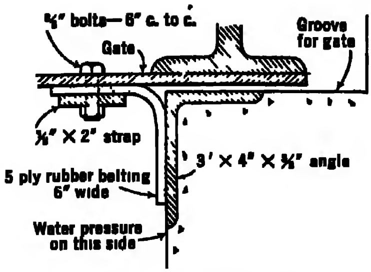


Fig 44 Side seal for stoney gate

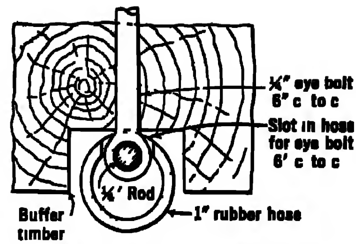


Fig 45 Hose type of seal for bottom of stoney gates

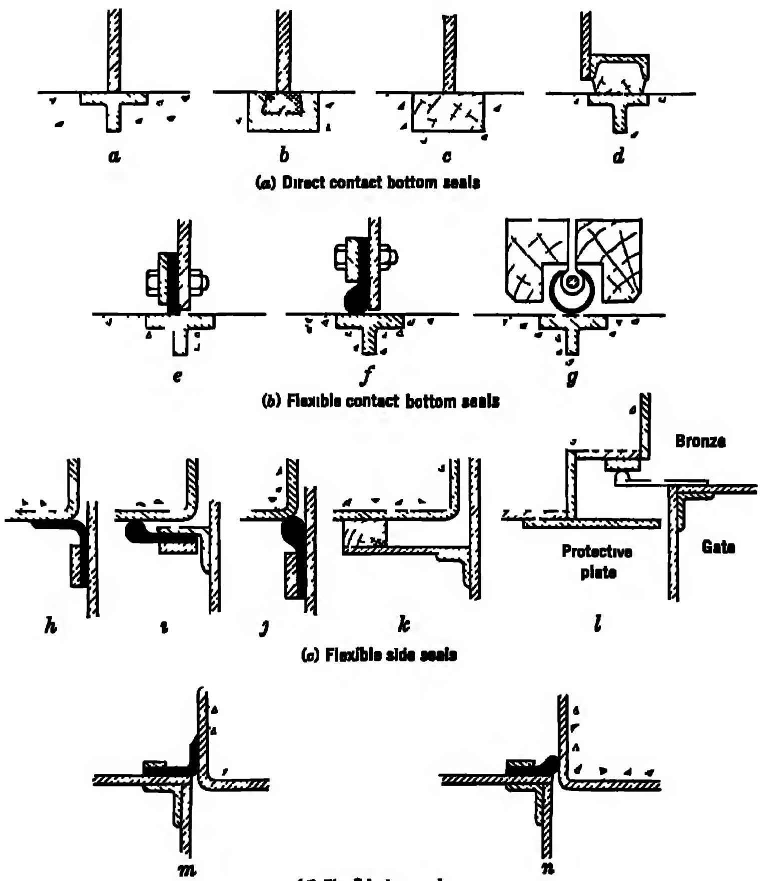


Fig 46 Typical forms of seals for crest gates

is a rubber hose inserted in a recess in the timber about one half the depth of the outside diameter of the hose, as shown in Figs. 45 and 46*g*. The hose is ordinary garden hose of 1-in. diameter. One-quarter-inch bolts with an eye at one end are put through the buffer timber on the bottom of the gate. The upper end of each of these bolts is threaded and fitted with a nut. At the lower end the eye is allowed to project into the recess in the timber mentioned above. These bolts are spaced about 6 in. center to center. Then

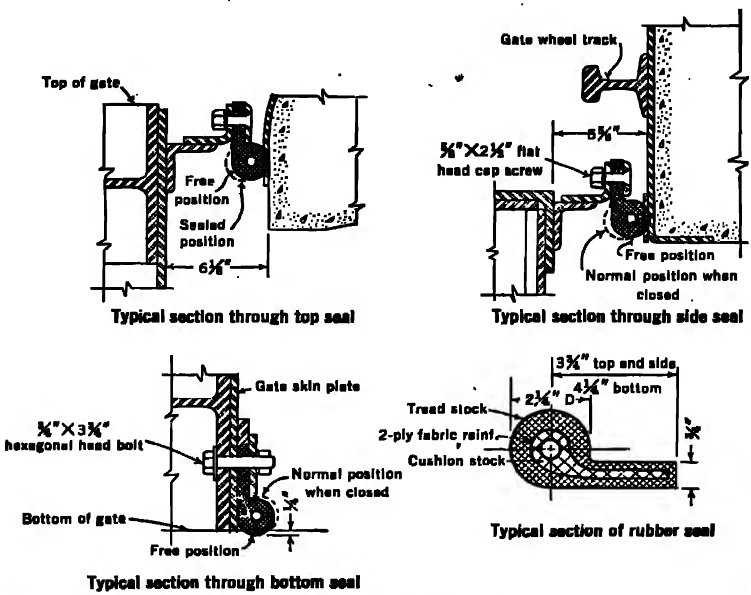


Fig. 47. Sealing device on head gates of Chickamauga hydro plant. (Tennessee Valley Authority.)

slots are cut in the upper part of the hose, and the hose is inserted in the recess in the timber so that the eyes of the bolts project through the slots in the hose. A ¼- or ⅜-in. rod is then run through the hose and through the eyes of the bolts. Then the nuts on the other ends of the bolts are tightened, thus securing the hose in position. The gate sill should consist of a channel or timber set in the concrete and carefully trued and leveled. If care is exercised in the work, this detail will secure an almost perfectly tight seal at the bottom of the gate. It is applicable also to taintor gates and slide gates.

Figure 47 shows the rubber-sealing device used on head gates of the Chickamauga plant of the Tennessee Valley Authority.

20. Force Required to Operate Gates. (*a*) *Sliding Gates*. The force required to lift a sliding gate is equal to the frictional resistance of the water pressure on the gate plus the weight of the gate. The force required to lower a sliding gate is equal to the friction less the weight of the gate. Starting

friction rather than moving friction determines the operating force. In some cases, additional lifting force is necessary for accumulation of ice or silt or for the weight of overflowing water.

The bearing friction is generally proportional to the water load on the face of the gate and is dependent on the type of bearing upon which the water load is supported.

Let F = the force required to operate the gate, in pounds;
A = the gross area of the gate, in square feet;
H = the head, in feet, from water surface to the center of A;
K = the coefficient of static friction; and
W = the weight of the gate, in pounds, corrected for submergence.

Then, to open the gate,

$$F = 62.5HAK + W$$

and to close the gate,

$$F = 62.5HAK - W$$

These equations apply to sliding gates used without filler gates. A filler may materially reduce the friction that must be allowed for.

In plain sliding gates, the coefficient of friction depends on the materials in sliding contact and the condition of their surfaces. When the same or similar metals are used for both the sliding and fixed surfaces, the surfaces are likely to seize when operated in sliding contact. Ferrous metals subject to rust may develop excessive friction due to the accumulation of rust or pitting where they are exposed to alternate wetting and drying. Recommended allowances for the coefficient of static friction between well-finished clean surfaces of commonly used materials are given in Table 2. Dry surfaces are considered in the table, since the lubricating effect of water is problematical where two surfaces are squeezed together tightly.

TABLE 2

RECOMMENDED ALLOWANCES FOR K COEFFICIENT OF STATIC FRICTION FOR SMOOTH-FINISHED DRY SURFACES

Steel on steel	0.6	Wood on metal	1.00
Steel on cast iron	0.6	Wood on wood	1.10
Steel on bronze	0.45	Rubber on metal	1.10
Bronze on bronze	0.45		

Stoney gates, caterpillar gates, and fixed roller gates all rest on some form of roller bearing, and the primary resistance to their movement is rolling friction. The coefficient of rolling friction is low and would not exceed 0.005 with reasonably good workmanship. However, the parts holding or containing the rollers may introduce considerable friction. In stoney gates, friction is produced in the bearing of the roller axles in the side bars or rollers. Likewise, the links and pins in the caterpillar roller train introduce a friction load. In fixed roller gates or wheeled gates, friction occurs in the contacts of the

closure seals and of the rollers and in the lubrication in the bearing spaces. The friction load that can be accumulated from these sources is dependent largely on the condition of the parts and is quite variable. For gates of this type, it is usual to allow for a load amounting to 6% of the water load on the face of the gate for the bearing friction.

The foregoing recommended values of K are applicable only if the intake gates are normally open. If the gates are likely to remain closed for a long

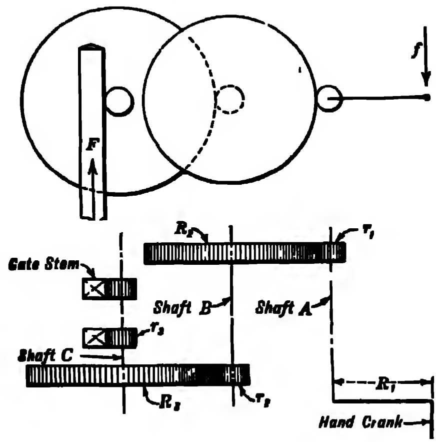


FIG. 48. Rack-and-pinion hoist with spur gears.

period, these values can be adopted if wood is used for one surface of contact; but, for bronze surfaces on rolled steel and on finished castings, or for bronze on bronze, the value should be 50% greater than the foregoing ones. For normally closed gates, rolled-steel and iron and steel castings should never be used except with wood or bronze, as they are likely to rust tight.

A factor of safety to allow for material obstructions under the gate when closing is seldom provided, because such obstructions are uncommon and, if they do occur, it is much better to remove them than to endeavor to cut through them with extra hoist capacity.

(*b*) *Caterpillar Gates.* According to E. B. Philips, the hoists designed by Philips and Davies for operating caterpillar gates are figured for:

(The rolling friction of the caterpillar rollers against the roller races, equal to 5% of the hydrostatic pressure against the surface of the gate) plus (the weight of the gate) plus (the vertical component of the water pressure on the inclined surface of the gate) minus (the buoyancy, or uplift, of the gate when closed.

Philips states also that, although 5% is used above for rolling friction, shop tests show that rolling friction under ideal conditions is less than 1%. This

factor of safety, together with 50% or more overload capacity of the hoist, provides for possible inaccuracies in the adjustment of the gates.

(c) *Wheeled Gates.* The force required to operate roller-bearing wheeled gates (see Figs. 26, 27, and 28), in which the pressure on the gate is transmitted through axle bearings to wheels that operate on gate guides, is given by the following equations. These equations apply directly, however, only where the wheels bear on vertical guides and the pull, F, is also vertical. Where the guide is at an angle to the direction of the pull, the force necessary to lift the gate is equal to F plus $P \tan \alpha$, where α is the angle between the pull and the direction of travel.

$$F = \frac{P}{R}(f_r + f_a r) + W$$

where F = the force required to raise the gate, in pounds,
P = the water pressure against the gate, in pounds,
R = the radius of the wheels, in feet,
r = the radius of axles, in feet,
f = the coefficient of rolling friction,
f_a = the coefficient of axle friction,
W = the submerged weight of the gate.

$$F_1 = \frac{P}{R}(f_r + f_a r) - W$$

where F_1 = the force required to lower the gate, in pounds.

The starting coefficient of axle friction f_a with plain axle bearings, has a value of about 0.3. If the axle has ball bearings f_a will not exceed about 0.01. The starting coefficient of rolling friction should be taken as 0.003.

In determining the size of hoist, the values of F found from the foregoing equations should be multiplied by a factor of safety of 2.

If watertightness is obtained by a belting strip, the force must be increased sufficiently to overcome the friction of the belting as explained in the following paragraphs on tainter gates.

(d) *Tainter Gates.* In the tainter gate the operating force has the advantage of leverage over the frictional resistance at the bearing. The bearings are usually of the plain cylindrical type arranged for grease lubrication. When the gate stands for a time under load, the lubricant is squeezed out from between the contacting surfaces and the coefficient of friction approaches the value for unlubricated surfaces. The operating force F required to overcome the bearing friction can be expressed as follows:

$$F = \frac{KPr}{R}$$

when P = total water pressure on gate,
K = coefficient of friction, as in Table 2,
R = radius at which F is applied,
r = radius of bearing.

Seal friction is dependent on the pressure exerted upon the seal contact and the coefficient of friction between the sealing surfaces. In flexible seals, the force exerted at the seal contact is usually derived from the headwater pressure, although mechanical springs have been used to increase the contact pressure. In rubber seals, closure is generally effected close to the headwater side of the contact area, and the headwater pressure should be considered as acting over the entire seal area. The less flexible wooden seal surfaces may have their line of closure at any place in the contact area, and, therefore, it is safer to assume for the estimation of seal friction that the headwater pressure acts over the entire contact area. The coefficient of friction between rubber and smooth metal surfaces is low as long as the contact is lubricated by a thin film of water. In gate seals, water is not retained between the contact surfaces and the coefficient of friction is relatively high. Coefficients of friction for materials commonly used for seals are included in Table 2.

21. Capacity and Efficiency of Hoists. Let

F = the pull or push to be exerted on the gate, determined as in Section 20;
f = the operating force applied to the hoist;
G = the leverage ratio of the hoist;
E = hoist efficiency (combined efficiency of gears and sheaves), expressed as a decimal;
e = efficiency of a pair of gears, bearings included;
T = torque required to operate hoist, in foot-pounds;
S = revolutions per minute of hand crank or motor;
s = speed of gate travel, in feet per minute;
H = horsepower required to operate the hoist;
R and r = radii of hand crank and gears;
p = pitch of worm or screw gears.

All dimensions are in feet and forces in pounds.

The leverage ratio, G, of the hoist is the theoretical ratio of the force applied to the hoist to the force exerted by the hoist on the gate, neglecting friction. It is also equal to the product of the leverage ratios of its component parts.

Leverage ratios are more easily understood by considering each shaft separately. Thus, the leverage ratios for the rack-and-pinion hoist in Fig. 48 are

$$\text{Shaft } A = \frac{R_1}{r_1}$$

$$\text{Shaft } B = \frac{R_2}{r_2}$$

$$\text{Shaft } C = \frac{R_3}{r_3}$$

and the leverage ratio of the hoist is

$$G = \frac{R_1R_2R_3}{r_1r_2r_3}$$

For the worm-gear hoist in Fig. 49, the leverage ratios are

$$\text{Shaft } A \quad \frac{2\pi R_1}{p_1}$$

$$\text{Shaft } B = \frac{R_2}{r_2}$$

and the leverage ratio of the hoist is

$$G = \frac{2\pi R_1 R_2}{p_1 r_2}$$

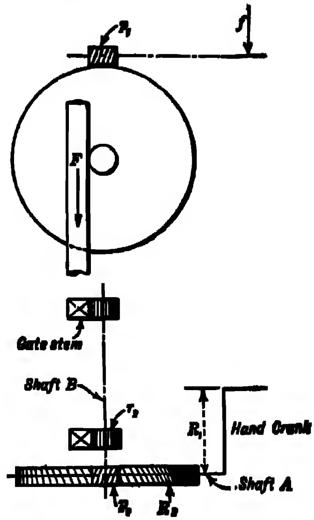


Fig. 49 Rack-and-pinion hoist with worm gear

In Fig. 50, the leverage ratios are

$$\text{Shaft } A = \frac{2\pi R_1}{p_1}$$

$$\text{Shaft } B = \frac{2\pi R_2}{p_2}$$

and the leverage ratio of the hoist is

$$G = \frac{4\pi^2 R_1 R_2}{p_1 p_2}$$

In Fig 51, the leverage ratios are

$$\text{Shaft } A = \frac{R}{r}$$

$$\text{Pulleys} = n$$

where n is the number of turns (4 in Fig 51) and the leverage ratio of the hoist is

$$G = nR$$

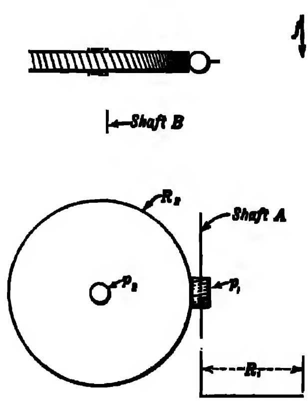


Fig 50 Screw-stem hoist with worm gear

The efficiency of a hoist is the ratio of the theoretical force required to operate the gate, with frictionless bearings and gears, to the actual force required. It is equal to the product of the efficiency of its component parts. Expressing it as a decimal, let the efficiency, including bearings and thrust collars, be

e_R = efficiency of a pair of spur gears;
e_g = efficiency of a screw gear;
e_W = efficiency of a worm gear.

Then, since the hoist of Fig. 48 has three pairs of spur gears, its efficiency is

$$E = e_R^3$$

And, since the hoist of Fig. 49 has one worm and one spur gear, its efficiency is

$$E = e_W e_R$$

And, since the hoist of Fig. 50 has one worm and one screw gear, its efficiency is

$$E = e_W e_S$$

A close determination of the efficiency, e, of the component parts of a hoist can be made only after a careful study of the details of such parts, the proposed method of lubrication, and the care to be given the hoist after installation.

E. B. Philips * gives the following approximate values of efficiency of single gears with bearings. These values are considered safe for properly maintained hoists.

	Plain Cast-iron Bearings	Bronze or Babbitt Bearings	Ball Bearings	Ordinary Roller Bearings
Spur gears with cut teeth	0.91	0.94	0.95	0.94
Spur gears with cast teeth	0.88	0.90	0.91	0.90
Bevel gears with cut teeth	0.90	0.92	0.94	0.93
Bevel gears with cast teeth	0.86	0.88	0.89	0.88

The efficiency of each steel cable sheave and its bearing is about 97%.

The efficiencies of screw and worm gears vary with the angle of the thread and ordinarily are lower than those of spur and bevel gears. Usual efficiencies of worm gears exceed 75% and this figure is recommended for preliminary calculations. For the actual efficiency, which can be determined only by consideration of many complicated factors, the reader is referred to the standard textbooks on machine design.

The force, f, required to operate a hoist is

$$f = \frac{F}{GE}$$

The torque required to operate a hoist is

$$T = fR_1$$

where R_1 = the lever arm of the force applied to the hoist.

The number of the revolutions per minute, of hand wheel or motor, necessary to operate the gate at a given speed is

$$S = \frac{Gs}{2\pi R_1}$$

where S = revolutions per minute;
G = leverage ratio of hoist;
s = speed of gate travel, in feet per minute;
R_1 = lever arm of force applied to hoist.

* Of Philips and Davies, machine builders, Kenton, Ohio.

The horsepower required to operate the gate at a given speed is

$$H = \frac{Fs}{33{,}000E}$$

in which H = horsepower;

F = pull or push exerted on the gate, as determined by Section 20;

s = speed of gate travel, in feet per minute;

E = over-all efficiency.

22. Gearing. Experience has shown the advantage of spur gearing over all other gears in both durability and efficiency. Its chief disadvantages is that it has not the great reduction or leverage ratio that screw and worm gears possess and that it, therefore, requires a greater number of gears and more space and weight of hoist.

The inefficiency of screw and worm gears, which require more work to be done in operating the gate, is not noticeable for very small gates or for hoists that are not hand-operated, particularly if, as is usual, operation is infrequent.

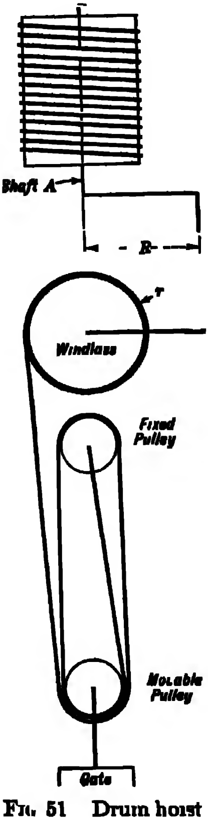


FIG. 51. Drum hoist.

To reduce the size of traveling hoists, worm gears frequently serve in conjunction with spur gears, as in Fig. 57.

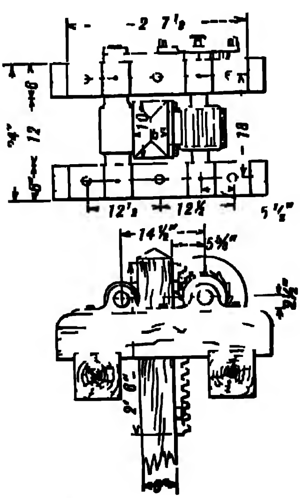


FIG. 52. Hand-operated single-stem gate hoist. 5000-lb capacity. (S. Morgan Smith Co.)

In some instances, worm gears are used in rack-and-pinion hoists, as in Fig. 49 but the preference for such hoists is for spur gears throughout, as in Fig. 53, as there is generally ample space for a spur-gear hoist.

The efficiency of spur and worm gears is materially increased if they are provided with roller bearings, but these are economically feasible only on the large gears. Both screw and worm gears of the better class of hoists are frequently submerged in an oil chamber, as in Figs. 54 and 57.

Hoists with cast spur gears are usually left exposed to the weather, but the teeth should not be shrouded or water and dirt will be held in them and

will freeze. Other classes of gearing should be protected from the weather unless vigilantly cared for. When exposed, gears require a heavy weather-resistant lubricant.

As obstructions are likely to lodge under the gate, the gearing of the hoist should be designed for the full power of the motive force.

When hoists are equipped with worm or screw gears, the friction is usually enough to eliminate the danger of the gates closing under their own weight.

23. Choice of Hoist. In some large installations one or two traveling gantry cranes operate all the head gates. For a large number of units, this

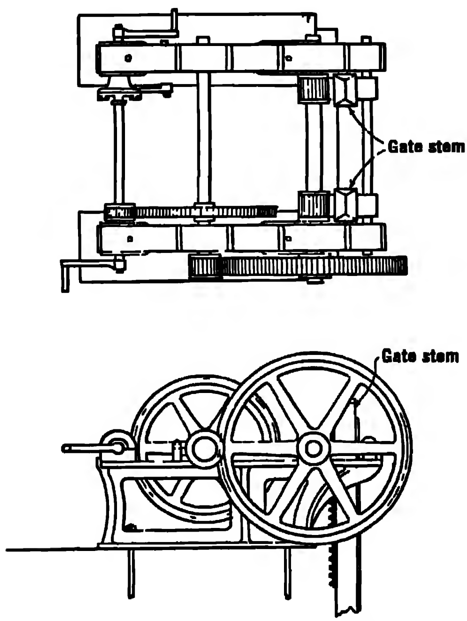


FIG. 53. Spur-geared hand-operated gate hoist. (Rodney Hunt Machine Co., Orange, Mass.)

system is cheaper than separate hoists for each gate but makes it impossible to open or close all the head gates in the minimum length of time. When opening and closing is of great importance, the additional expense of individual hoists may be justifiable. For each development an economic study should be made to determine the advisable design.

At some dams the head gates are merely extra guard gates, as at Conowingo, where there is a butterfly valve at the entrance of each scroll case. In such a situation a traveling gantry is all that is justifiable for operating the head gates.

Mechanisms of various types are used for the operation of intake gates. The most common ones are listed below and described in succeeding sections.

Rack-and-pinion hoists (Fig. 53)
Screw-stem hoists (Fig. 54)
Drum hoists (Fig. 56)
Hydraulic hoists (Fig. 60)

Of these, the drum hoist is used exclusively for those gates that will close by their own weight, such as the taintor, caterpillar, wheeled, and stoney

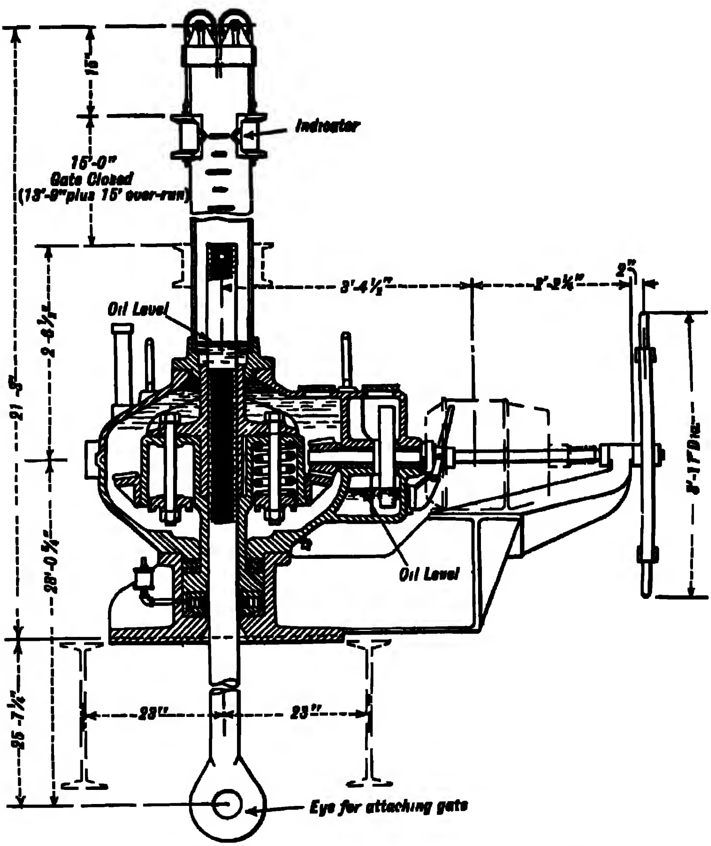


FIG. 54. Screw-type gate hoist at Spiers Falls, N. Y., for head gate 18 ft wide, 12 ft high under 30-ft head. Lifting capacity 150,000 lb, downward thrust 50,000 lb. (M. H. Treadwell Co., Manufacturers.)

gates. For gates that require a positive thrust to close them, the choice lies among the other types.

For very small gates and all gate valves, screw-stem hoists are preferable. For the larger sizes, screw-stem hoists are more expensive and, if they are hand-controlled, the work required to operate them is excessive on account of the low efficiency of the gears. They require very fine adjustment of gate

travel. Any extensive movement of the stem out of alinement during operation will bind the gear, decrease the efficiency, and cause excessive wear.

Rack-and-pinion hoists are more rugged and require less protection and care. They can be used with stems composed of structural-steel members. On the other hand, they occupy greater space on the intake and are limited as to capacity.

Both types are well adapted to motor control. For hoists between 10- and 30-ton capacity, the rack-and-pinion type seems slightly preferable. The screw-stem type is used exclusively for very large capacities.

FIG. 55. Typical floor stand for hand-operated small screw-stem gate. 35,000–50,000 lb capacity (Rodney Hunt Machine Co., Orange, Mass.)

Hydraulic hoists are used extensively. Their chief advantage is simplicity of construction. On the other hand, they require a number of auxiliaries, such as pumps, tanks, valves, and piping. The fluid for transmitting the pressure should in general be oil, as water may freeze or cause corrosion. Hydraulic cylinder hoists are quite well adapted to very large capacity.

24. Rack-and-pinion Hoists. The following are two of the many types of rack-and-pinion hoists on the market:

Single-stem, lever hoist (Fig. 52).
Double stem, spur-geared hoist (Fig 53).

The lever hoist, in which one end of the lever is frequently a socket wrench fitting on the same shaft as the pinion that operates the rack on the stem, is used only for small capacities.

Generally all the teeth of the racks and gears are cast and not cut, because of the slowness of operation and the exposure of such hoists to the weather. They are usually of cast iron except that, for the larger capacities, the top sections of the racks, which have the greatest duty, and the pinions engaging them are preferably made of cast steel. The tendency is to omit all shrouding on the gears, particularly in cold climates, where the hoists are exposed, to preclude the accumulation of ice and dirt, which may cause rupture of the teeth.

If the hoists are to be motor-operated, the high-speed portion should have cut-steel gears, which should be boxed or housed to protect both gears and motor.

On account of the low efficiency of worm gears and the greater protection and attention they require, spur gears are more frequently used for rack-and-pinion hoists, particularly for the larger sizes, unless lack of space or other special conditions demand worm-gear hoists.

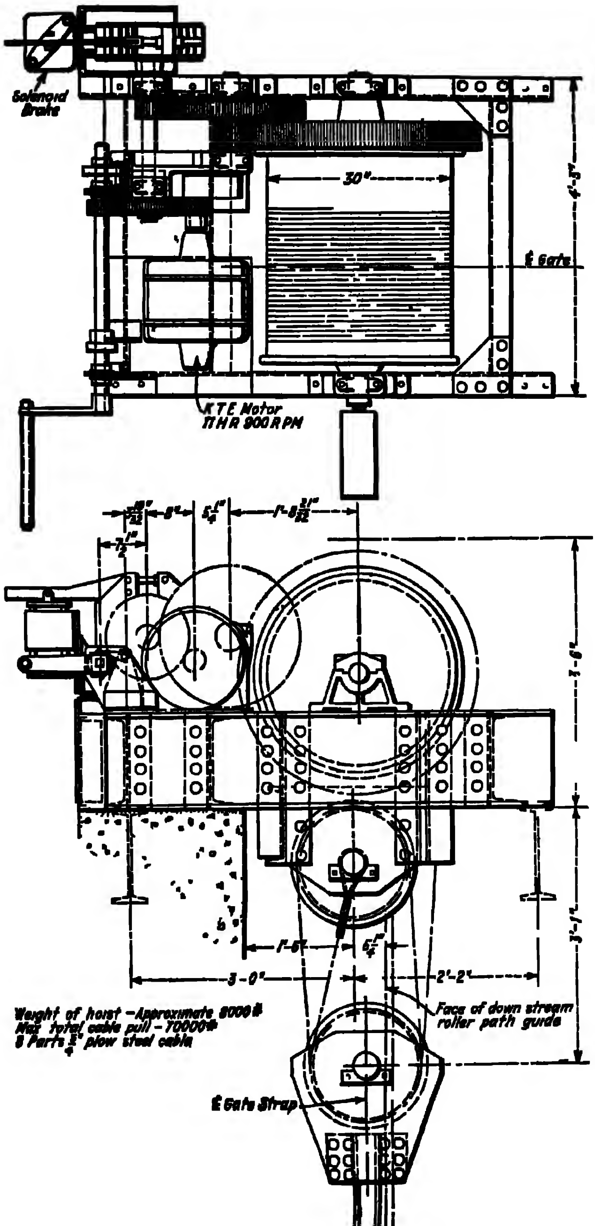


FIG. 56 Drum-type gate hoist as made by Philips & Davies Kenton Ohio

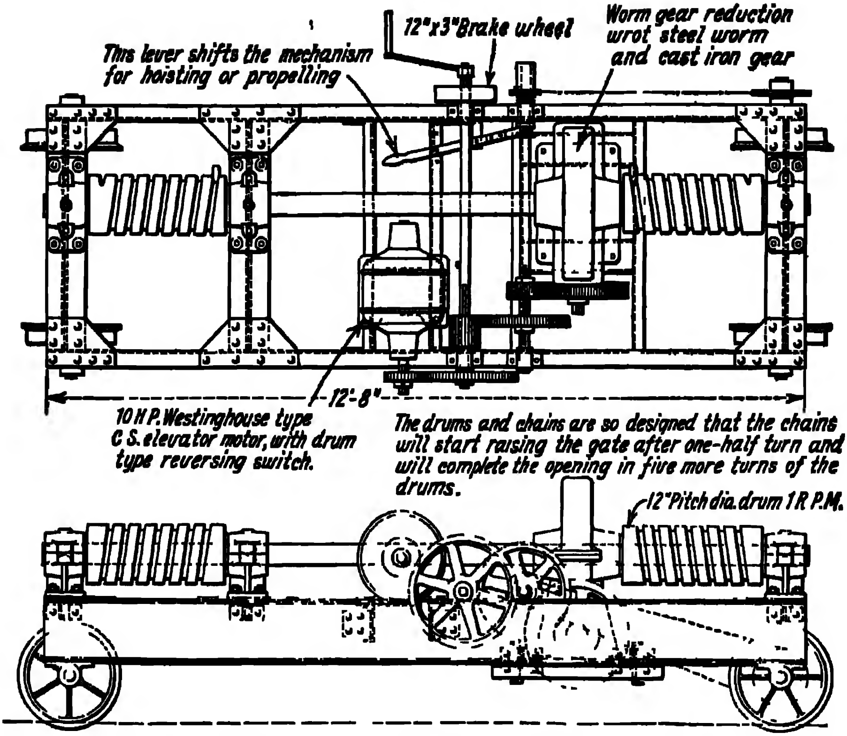


FIG. 57. Movable hoist for taintor gates, Kerckhoff Dam. (San Joaquin Light and Power Corp.)

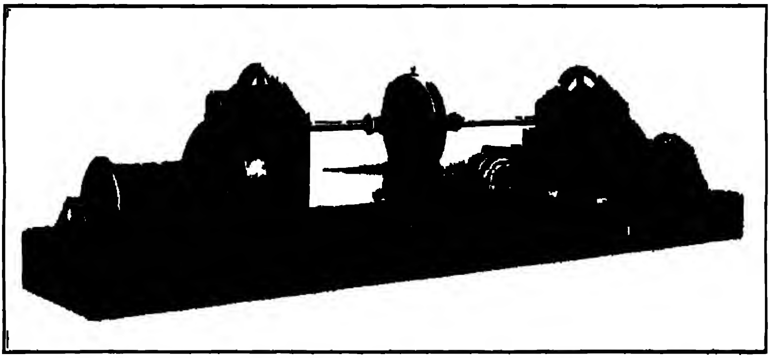
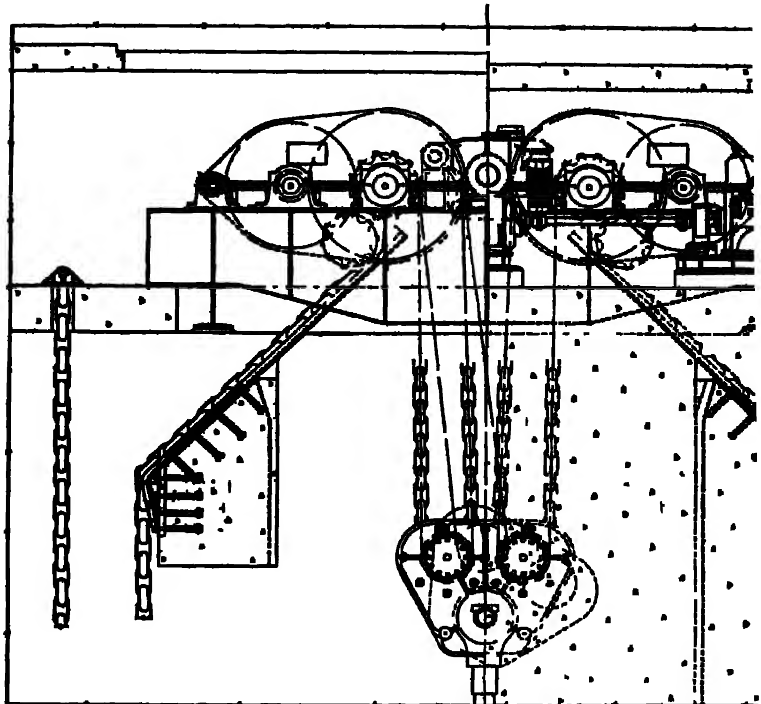
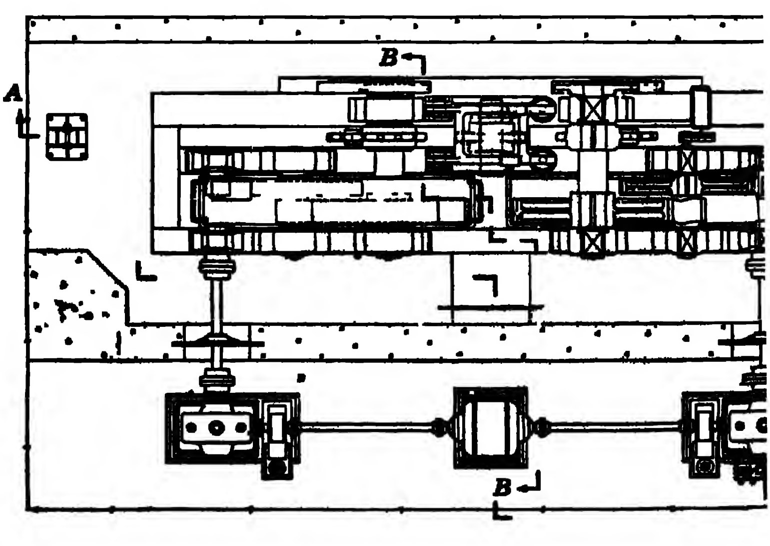


FIG. 58. Typical individual taintor gate hoist. (Philips and Davies Co.)



Section *A–A*



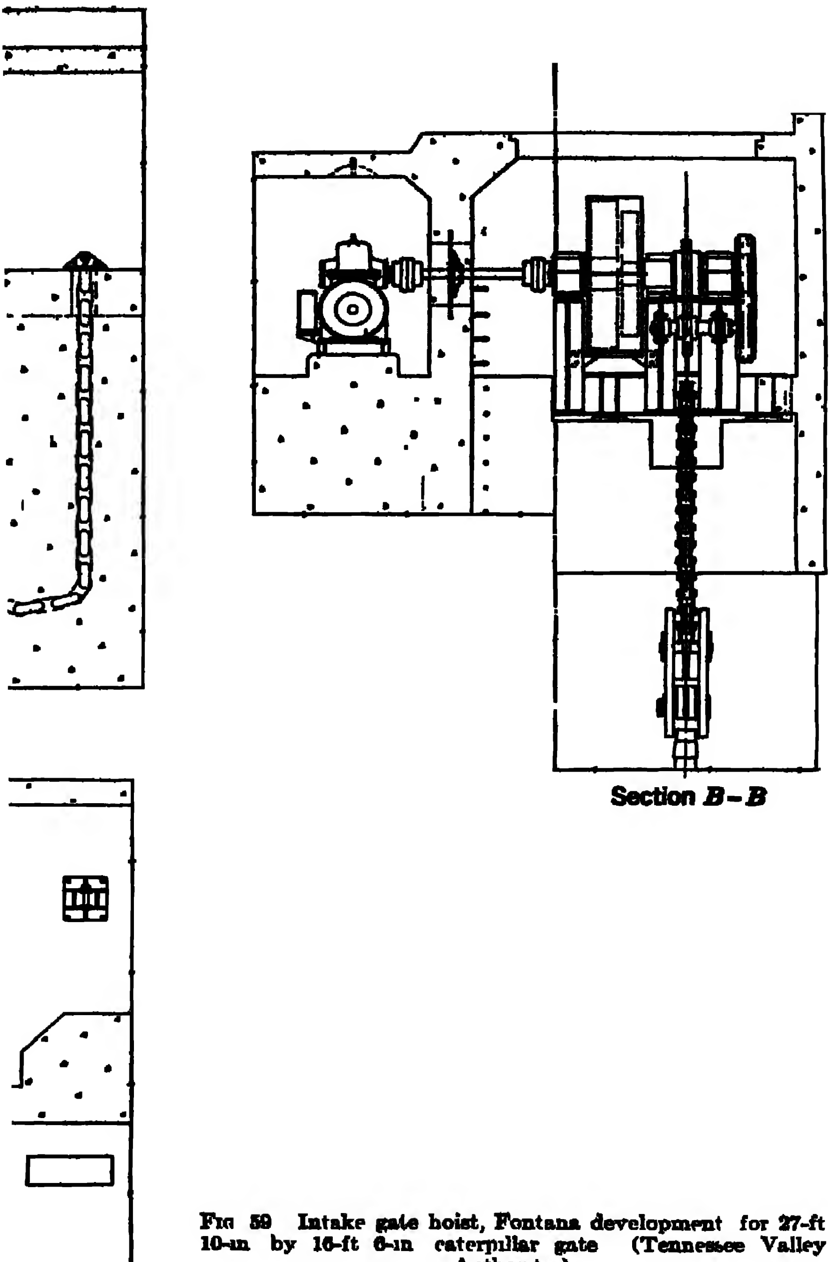


Fig 59 Intake gate hoist, Fontana development for 27-ft 10-in by 16-ft 6-in caterpillar gate (Tennessee Valley Authority.)

Frequently, provision is made for attaching the operating cranks to a higher-speed shaft for speedy operation during light loads. Motor operation is supplied by installing a greater gear reduction, and a clutch should be included for disengaging this additional gearing for hand operation in case of failure of the motor.

25. Screw Hoists. A typical screw hoist of large size is shown in Fig. 54 and small ones in Figs. 23 and 55. The large hoist is completely protected from the weather, even possessing a hood over the rising stem. Hoists of this type usually have roller bearings under the thrust collar to reduce the friction when lifting. If much force is required to close, roller bearings are provided above the thrust collar also.

To insure the greatest gear efficiency, the screw should be designed so that the gate will be almost able to overhaul the gearing and start closing under its own weight.

26. Drum Hoists. Figure 56 shows a drum type of hoist intended for the operation of a caterpillar gate; but very similar drum hoists are used for stoney and other gates capable of closing under their own weight.

Figure 57 shows a type of hoist particularly adapted to the operation of taintor gates. The taintor gate is usually raised by means of two cables or chains attached to the front lower corners of the gate. When the gate is raised the chain or cable winds up in the grooves on the drum, as illustrated in the figure. In some taintor-gate hoists, the drum consists of two truncated cones with the small end of each cone near the ends of the shaft. The chain starts to wind up at the small ends of the cones and thus a greater torque is obtained in starting.

Drum hoists may be equipped with chains or steel cables. Chains are usually adopted unless multiple sheaves (Fig. 56) are necessary. Figure 59 shows the hoist for the 27 ft 10 in. × 16 ft 6 in. caterpillar intake gate of the Fontana development of the Tennessee Valley Authority. There are two of these gates and hoists with a third intake blanked off. Each gate hoist normally exerts a pull of about 470,000 lb. Normal hoist speed of the gate is 3 ft per minute.

27. Hydraulic Hoists. The hydraulic hoist, as in Fig. 60, consists of a cylinder in which a piston connected to the gate lifts and lowers the gate by means of hydraulic pressure. The pressure is provided by pumps using oil as a fluid. Pressures as high as 1000 lb per sq in. have been used, but experience has shown that about 400 lb is preferable.

The piston packing, particularly under heavy pressures, is seldom tight enough to hold the gate open, and a latch or some other means must be used to support the open gate. For this and other reasons, hydraulic hoists are not particularly adaptable to remote control.

A fairly typical low-head installation * is illustrated in Fig. 60, which shows the gates for the Vernon extension units. Each of these units has a capacity

* From a description written by E. A. Dow for spring meeting of the American Society of Mechanical Engineers, 1929.

of 5000 kw under 34-ft head and discharges about 1800 sec-ft. Each has two gates 17 ft 6 in. high by 15 ft 9 in. wide, operated by 28-in. oil cylinders.

In line with the New England Power Company policy, these gates are designed to close in an emergency under full head with no back pressure, so that it is possible to shut off the flow with the turbine gates open or with the

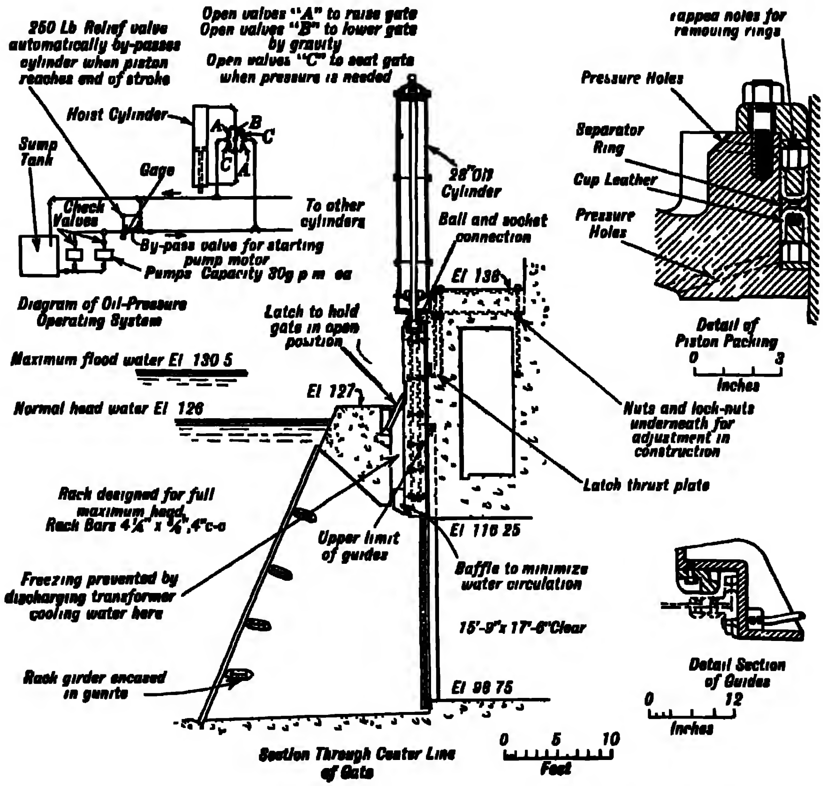
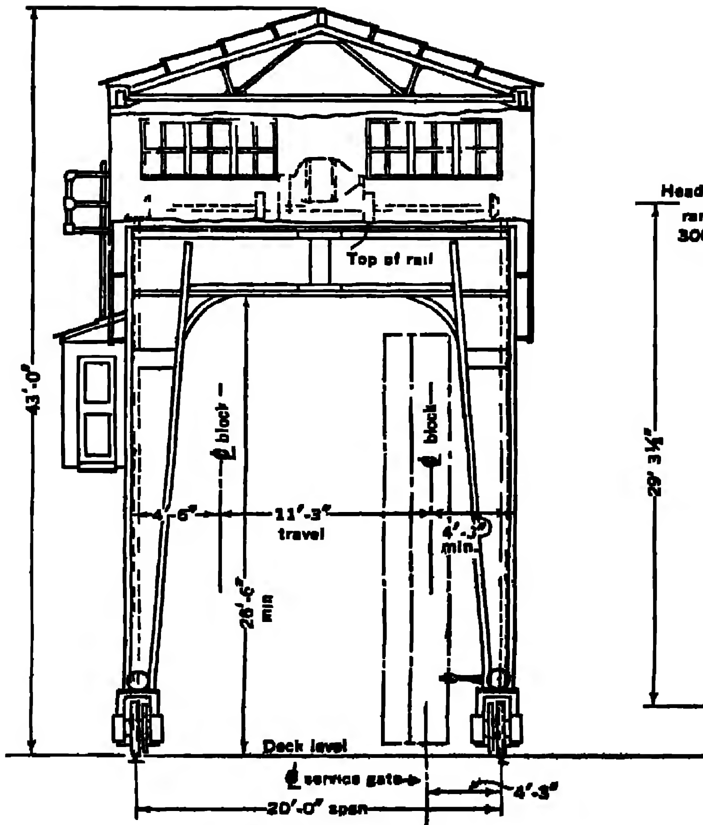


FIG 60 Hydraulic hoists of the Vernon plant, New England Power Co.

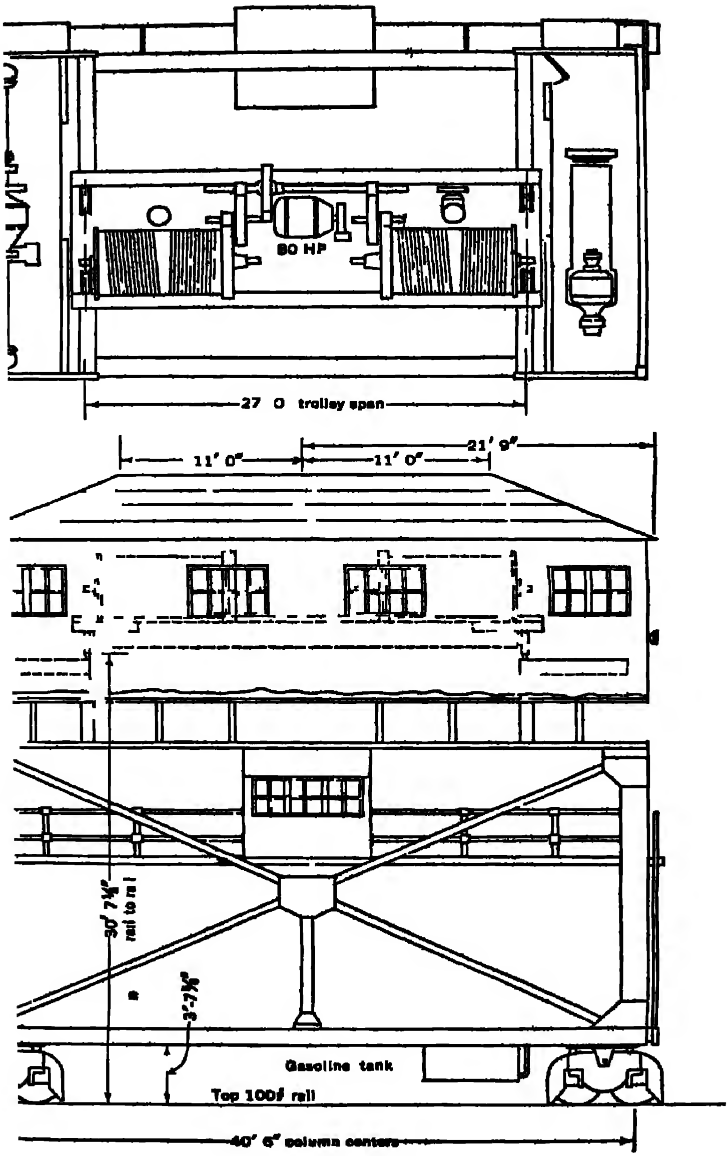
turbine entirely wrecked To aid in closing under emergency conditions, the gates were made very heavy (120,000 lb each), being constructed of a structural-steel frame filled with concrete. No skin plate was used. The weight of the gates plus the thrust of the hoists is sufficient to insure closure with a coefficient of friction of gate on guides of cell over 0.75. They are opened with pressures equalized by means of filler gates. These gates are of the plain sliding type and have seats of structural steel bearing on cast-iron frames

28. Motive Power. Gate hoists can be operated by hand, by electric motor, or by hydraulic pressure



End view and part section

FIG 61. Sixty-ton traveling gantry crane for operating] de]



…s at Conowingo hydroelectric development, Maryland (Phila-
…tric Co)

For hand-operated gates the crank effort required to start the gate usually diminishes rapidly as the gate is lifted. This effort in starting should not exceed 40 lb for gates normally closed and 30 lb for gates normally open. These values will correspond to about 20 lb for the effort after the gate is started.

The most convenient radius of the hand crank is about 15 in. with shaft about 3 ft above the floor. The radius of hand wheels varies considerably among manufacturers. The advantage of a hand wheel is that it can be spun rapidly under light loads. Cranks are more often used than hand wheels, and in many instances provision is made for changing the crank to a high-speed shaft when the load is light.

When the hoist is of such large capacity that hand operation becomes tedious, or when remote control becomes necessary, an electric motor is supplied. However, if practicable, the hoist should always be equipped with emergency hand operation for use if the motor control should fail.

Alternating- or direct-current motors can be used for gate hoists, the choice depending on the kind of current adopted for the other station auxiliaries. Ordinarily, variable-speed, high-starting-torque motors are required, as the type of hoist and gate is rarely such that the motor can attain full speed before being loaded. Constant-speed motors have been used successfully with drum hoists where the slack in the chain or cable, when the gate is closed, is sufficient to allow the motor to attain speed before the gate starts to lift; but this arrangement limits the adaptability of the motor. As an example, if such a gate were opened but slightly in order to fill the conduit gradually, it could not be opened further without first closing to slacken the cable so as to permit the motor to start.

Variable-speed d-c motors are series-wound. Variable-speed a-c motors can be either the high-resistance, squirrel-cage type or the slip-ring type.

Successful motor operation of gate hoists requires accurate control at the end of the gate travel, particularly if overtravel beyond the desired point would jam the gears and cause damage. Almost every type of gate provides ample leeway at the end of travel when opening; but, except for drum hoists, which can exert no direct downward thrust, the permissible overtravel at the end of the closing operation is very small.

The gate has a tendency to travel a short distance after the current is cut off from the motor, because of the inertia of the moving parts. Therefore, when the permissible overtravel is limited, the speed of operation at the end of gate travel should be reduced to facilitate stopping at the proper time. A reduction of speed at the end of gate travel is accomplished by means of hand controllers. Such controllers are difficult to operate from a distance, and hence for remotely controlled hoists, considerable permissible overtravel should be provided and the controller for reducing speeds should not be used. Controllers are usually furnished with a number of speeds, forward and back, although in many cases the danger from overtravel may be in only one direction. Controllers are not required for closing with a-c squirrel-cage motors

or d-c series motors if, as in the case of sliding gates, the load is greatest at the end of gate travel, because for these types of motors the speed is slowest when the load is greatest. Slip-ring a-c motors, on the other hand, are variable in speed only when starting and so would require controllers for stopping.

Limit switches should be provided for all installations. Such switches are essential for remotely controlled hoists, to throw off the current and stop the motor at the proper time. With hand control, they are very desirable to prevent overtravel due to carelessness of the operator. They should be adjusted to the inertia of the moving parts so that the continuance of gate travel after the current is off will not exceed the limit of permissible travel and jam the gears.

Solenoid brakes should be furnished for those gates that may overhaul the gears and close by their own weight. Such brakes are designed to set automatically when the current is cut off the motor. They are also useful to reduce the length of travel due to inertia, as previously explained.

In addition to a limit switch, a friction drive consisting of a fabric disk held by springs between metal disks is often advisable. This type of drive is most useful if there is danger of an obstruction lodging under the gate and causing jamming before the limit switch stops the hoist. However, the danger from such obstructions at intakes protected by racks is not great, and a friction drive adds one more part to maintain.

Overhead trolleys are considered best for motor-operated traveling hoists. Plug-and-receptacle systems have been used but are frequently a source of trouble, particularly if the removable connecting cable may be run over when moving the hoist.

An electric motor may be provided for each hoist; or, by means of a line shaft with clutches, a single motor can drive two or more hoists successfully. A motor with each hoist is the most desirable arrangement but usually the most costly.

29. Traveling Hoists and Gantries. It is common practice to mount drum hoists on wheels, to travel along the top of the intake and operate each of the head gates. Such hoists are shown in Figs. 57, 61, and 62.

There are some objections to traveling hoists, as in some conceivable emergencies it might be very important indeed that the head gate controlling some particular unit or penstock be put down as promptly as possible. If the hoist were far removed from the gate affected, it would take quite a while to unhook it from the gate to which it was then connected, move it to the gate to be operated, attach it, and lower the gate. By the time the hoist was in position, untold damage might be done that might have been avoided had the gate been hooked up to a hoist ready for instant operation. To overcome this objection, latches have been arranged at some installations, so that in an emergency the head gates can be dropped into position.

Where traveling hoists are used, there is also a greater temptation to normally leave all gates up with headwater pressure riding on the wicket gates of the turbines so that units can go on the line promptly when in demand.

The leakage through the wicket gates when units are not operating may be tremendous as a result of this practice. Consequently, this fact should be considered in determining whether traveling hoists or individual hoists are more economical for any particular project.

The engineer charged with the responsibility for a project must weigh the saving in first cost effected by using one hoist for several gates against the assurance that an individual hoist for each gate gives. For moderate- and high-head developments of the first magnitude, it is believed that usually it

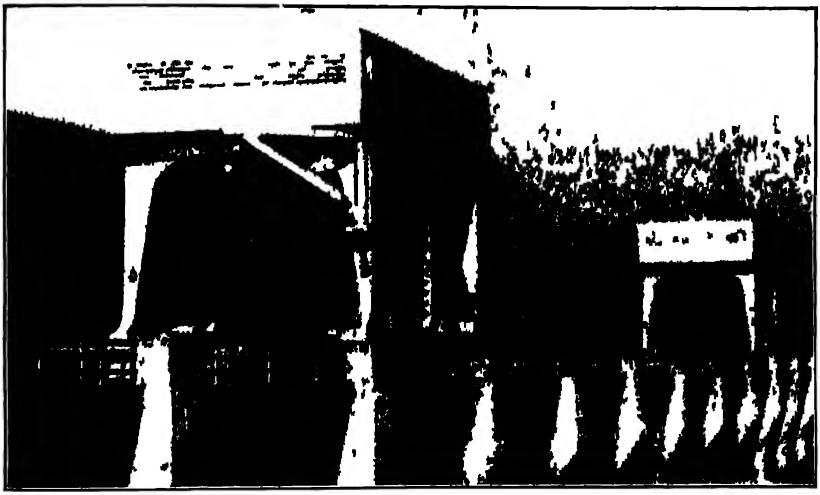


FIG 62 Intake gantry cranes at Chickamauga development (Tennessee Valley Authority)

will prove worth while to install an individual hoist for each gate For gates without hand operation more than one source of power should be provided

Traveling hoists are usually mounted on a simple car or truck equipped with a track, as in Fig 63 However, in some instances, where the intake has a superstructure, a powerhouse type of crane has been provided not only to operate the gates but also to handle trash racks, raking devices, and stoplogs In the absence of a superstructure, a gantry crane has served this purpose

30. Stoplogs. Provision must be made for unwatering the intake gates for inspection and repairs In some cases elaborate emergency gates that can be quickly operated have been installed, but such methods are too special for detailed description It is considered that properly designed gates should not require frequent unwatering Consequently, it is usual merely to install stoplogs for that purpose

If the span is not too great, stoplogs generally consist of timbers guided into place by means of a vertical slot at each side of the opening above the gates If the opening to be closed is wide and deep, steel I beams, with or without timbers bolted to them can be used, as in Fig 64 These give both

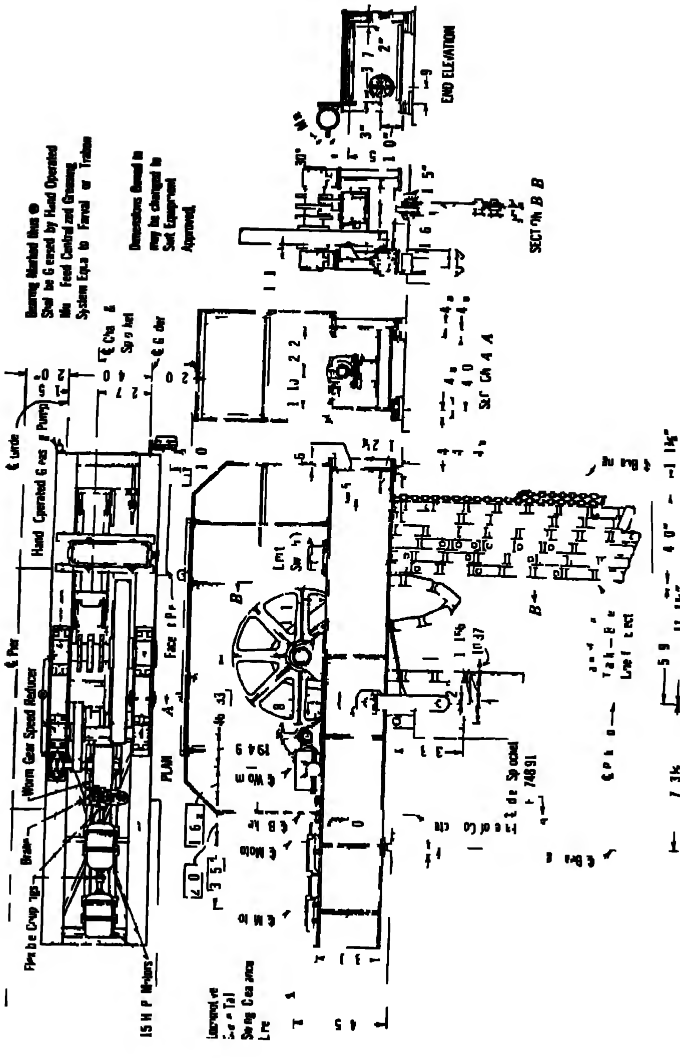


FIG 63 Gate hoist Emsworth Dam Pennsylvania (U S Engineer Office Pittsburgh Pa)

the necessary strength and sufficient weight to facilitate installation by overcoming the buoyancy of the wood.

Stoplog grooves are sometimes provided upstream from the racks, but much more frequently they are located just upstream from the gate, as in Fig. 6. In such cases, unless the forebay can be unwatered, the piers between the racks are allowed to project upstream from the racks, as in Fig. 16, so that a bulkhead can be put in place on the extremely rare occasions when it becomes necessary to unwater the racks.

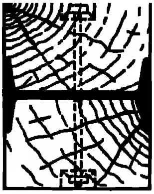


Fig. 64. Typical steel wood stoplog.

Unless special care is taken to fill the sides of the grooves with exceptionally smooth concrete, the stoplogs cannot be put in place and made tight without some difficulty. Consequently, wood or steel seats similar to those used for sliding gates (Figs. 23 and 25) are frequently provided.

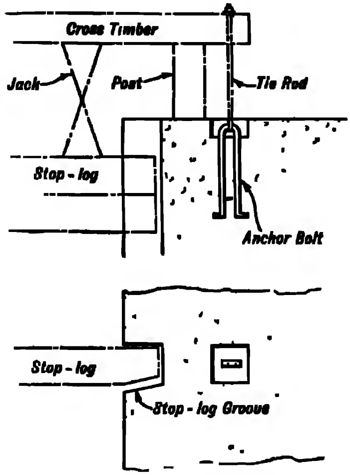


Fig. 65. Arrangement for jacking down stoplogs.

Where wooden stoplogs are adopted, it is difficult to overcome the buoyancy of the wood without special provisions. Figure 65 shows a permanent anchor bolt imbedded in the concrete at each groove. Fastened to this is a temporary cross timber that serves as a support to jack down the logs.

Figure 66 shows an arrangement used where the buoyancy of the logs is insufficient to bring them to the surface and where they must be removed against water pressure. It consists of a bolt passing through a notch cut into one end of each log, which may be engaged by a hook on the end of a rod to afford a means of pulling the log to the surface.

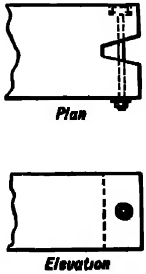


Fig. 66. Special arrangement to facilitate removal of stoplogs.

31. Air Inlets. Air inlets are particularly important just back of head gates at the entrance to a penstock or pipeline, because if they are omitted and the pipeline is suddenly drained when the head gate is closed, collapse of the pipeline is likely to follow.

The ratio of square inches of air inlet to square feet of gate opening has varied all the way from 1.5 to 9.5 sq in. of inlet per square foot of gate opening. In a few cases it is very much greater; for example, in the Rocky River hydroelectric plant, Connecticut (Fig. 21), the ratio is 16.1 sq in. per sq ft of gate opening.

As some serious accidents and failures of pipelines and penstocks have been due to inadequate venting, it is desirable to check the adequacy of air inlets that may be installed within the contemplated structures. If vents are inadequate, it is possible to increase their area by enlarging the structure.

The following equation for flow of air, applicable to ordinary cases, as explained later, is derived from an article by M. L. Enger and F. B. Seely: *

$$Q = 348cF\sqrt{P}$$

Carman and Carr's equation for the strength of steel pipe is †

$$P = \frac{50{,}200{,}000}{s}\left(\frac{t}{d}\right)^3$$

Combining these two equations, we have for the safe area of the air inlet

$$F = \frac{Q\sqrt{s}}{2{,}460{,}000c}\left(\frac{d}{t}\right)^{3/2}$$

In the above formula,

Q = flow of air through inlet, in cubic feet per second;
c = coefficient of discharge through air inlet;
F = area of air inlet, in square feet;
P = safe difference in pressure between inside and outside of pipe, in pounds per square inch;
t = thickness of steel pipe, in inches;
d = diameter of steel pipe, in inches;
s = a factor of safety against collapse of pipe.

For pipes buried in earth, a value of $s = 5$ should be used; for pipes on saddles, $s = 10$ is a safe value.

Values of $c = 0.5$ for the ordinary type of air-inlet valves, and $c = 0.7$ for short air-inlet pipes, are conservative.

Owing to adiabatic expansion, the above formula becomes quite inaccurate (but on the safe side) when the difference in pressure between the inside and outside of the pipeline, P, exceeds 5 lb. However, this is not particularly important, as the permissible difference in pressure is much less than this for any but very small pipelines. For instance, the collapsing-pressure equation shows that a 54-in.-diameter pipeline with a thickness of steel of ¼ in. would collapse at a difference in pressure of about 5 lb per sq in.

* "Vents on Steel Pipe Lines," *Eng. News-Rec.*, Vol. 69, p. 594, 1914.
† *Bulletin* 5, University of Illinois Experiment Station. See also reports of other experiments on collapsing pressure, including E. E. Stewart, *Trans. A.S.C.E.*, Vol. 27, p. 730, 1906.

As an example of the application of the formula, let

Q = 500 sec-ft, flow of air through the inlet because this equals the maximum flow of water in the conduit;
c = 0.7;
t = 0.5 in.,
d = 100 in.,
s = 10.

Then from the above equation

$$F = \frac{500\sqrt{10}}{2,460,000 \times 0.7}\left(\frac{100}{0.5}\right)^{3/2} = 2.6 \text{ sq ft}$$

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CHAPTER 28

CONDUITS

1. Types of Conduits. After the water has passed the intake at the dam, it enters the conduit and is carried from there to the turbines in the powerhouse. The general types of conduits used in hydroelectric practice are as follows:

1. Canals.
2. Flumes.
3. Steel pipe.
4. Wood-stave pipe.
5. Concrete pipe.
6. Tunnels.

The conduit system may be divided into two parts, namely the "high-line" or nonpressure conduit and the "penstock" or pressure conduit. Beginning

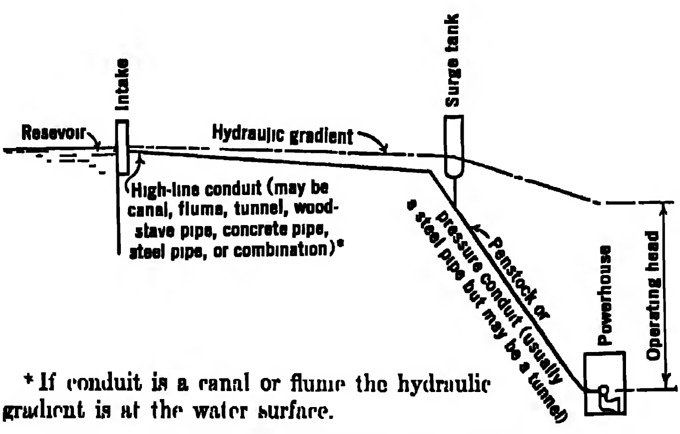

FIG. 1. General application of various types of conduits.

at the intake of the dam, the high-line conduit is that portion which follows the hydraulic gradient or slightly below it, and the pressure conduit is that portion which continues the flow to the turbines well below the hydraulic gradient. A typical make-up of the conduit system is indicated in Fig. 1.

Canals, flumes, and closed conduits, such as nonpressure tunnels, wood-stave pipe, and concrete pipe, are used for the high-line portion of the conduit system. Occasionally steel pipe has been employed, but it is generally too expensive to be desirable. If a closed conduit is used for the high-line con-

duit, especially if it is a wood-stave pipe, a moderate amount of pressure in the conduit is desirable, say up to a maximum of 30 ft of head to prevent the rotting of the wood pipe due to lack of saturation.

For the pressure conduit the choice is much narrower. Because of the high pressure to which this portion of the conduit system is subjected, the choice, for high- and medium-head plants, lies between properly protected pressure tunnels and steel pipe. Most penstocks are steel pipe. For medium-head plants, wood-stave and concrete pipe have occasionally been used. For low-head plants the conduit is commonly in the dam structure.

The principles of the different types of conduits are discussed in succeeding chapters as follows: canals in Chapter 29, flumes in Chapter 30, steel pipe in Chapter 31, wood-stave and concrete pipe in Chapter 32, and tunnels in Chapter 33.

2. Location of Conduits. The location of the conduit system is sometimes made by methods similar to those used in railroad location work. The locating engineer decides the most feasible line and establishes the alinement in the field.

For important projects, and where there is much topographic relief, considerable reconnaissance work and mapping is done for the entire area which may come within the scope of the project. With the necessary field information, an office study is made to determine the most desirable and economical conduit location The proposed line is then staked out in the field to substantiate the paper location and to obtain the final location of the conduit. The choice of location is discussed in Section 7 of Chapter 7.

3. Limitations and Relative Advantages. In the choice of a conduit, topographical and economic considerations will generally govern. A thorough investigation should be made of the feasibility of using various types of conduit for the project under consideration, and comparative estimates of cost should be prepared for the various types of conduit and for alternative alinement.

Thus, for a certain project, it might at first appear a comparatively simple and cheap proposition to conduct the water by means of an unlined canal around the brow of the hill to a point just above the powerhouse where the intake for the penstocks would be located. Further investigation and borings along the line of the work might show, however, that the material through which the canal would pass was a coarse gravel which would cause an excessive seepage, and that consequently it would be necessary to line the canal. With this additional cost it is found, as an assumption, that the total cost of the canal line is greater than that of an alternative tunnel through ledge rock under the hill.

When the country is so steep and rugged that a conduit could not follow the hydraulic gradient, canals are, of course, ruled out of consideration, and the choice lies between steel pipelines, wood-stave pipelines, reinforced-concrete pipes, and tunnels, or a combination of two or more types.

The conduits must be considered in connection with the other features of the project, as the various parts of the system are interdependent. Thus, it might pay to change the location of a dam site in order to shorten a conduit. On some projects the required length of conduit is found to be so expensive that it pays to break up the project into several smaller ones having a very much shorter total length of conduit. The extra ponds thus take the place of the conduit. Generally speaking, a steep slope of the river and a high load factor favor the use of tunnels and pipelines. A flat country and a low load factor favor the elimination of conduits, or their reduction to the minimum and the location of the powerhouse as an integral part of the dam.

In cold climates, both canals and flumes are objectionable if of great length, on account of the problems arising from the formation of ice. Frazil ice,* forming in the canal before it is covered with an ice blanket, often blocks the racks or the turbines. This trouble ceases when the canal is frozen over. Flumes, on the other hand, are frequently designed for such high velocity that the surface never freezes, and frazil ice trouble may be expected. Pipes often give trouble from freezing. In many cases of exposed steel pipe, a coating forms on the steel inside the pipe and reduces the area somewhat. This ice coating, when it becomes loosened in warm weather, usually passes through the turbines without injurious results but sometimes blocks the turbine gates completely. Concrete and wood are better insulators than steel, and freezing in concrete and wood-stave pipe seldom occurs.

Unlined tunnels in rock may be used if the material is stable without support and if they are deep enough in the ground so that the weight of the overburden is sufficient to balance the internal pressure. Where the material is unstable or where the rock shows excessive leakage, tunnels are lined with concrete and reinforced with steel if the depth is insufficient. Unlined tunnels will often compete successfully with other types of conduits. However, the cost of lined tunnels is so great that they are adopted only to meet conditions which prevent the economic installation of other types of conduits.

4. Economics of Conduits. High velocity, with resultant small area and small size of conduit, makes for cheapness in first cost but results in high friction loss and decreased head and power output. Since minimum cost can be obtained only at a sacrifice of output, and since maximum output can be obtained only at an increase in cost, there is always, for every project, one size of conduit which, theoretically, will result in the greatest economy of design.

Hence, the theoretical best velocity in conduits is fixed by the principles of maximum economy. A typical investigation of the economics of design for the Colorado River Aqueduct is presented by Julian Hinds.†

However, two practical considerations influence the choice of velocity.

(*a*) *In canals in earth* the velocity must not be high enough to cause scour or low enough to allow plant growth or deposits of silt. (For permissible velocities in earth canals see Sections 6 and 7 of Chapter 29.)

* For definition see Section 6, Chapter 27.

† See Refs. 2 and 3, Section 10.

(*b*) *In penstocks* the velocity is influenced to a large extent by consideration of turbine regulation. A high velocity, though it may prove more economical, may require such a slow-moving governor to prevent excessive water hammer (see Chapters 34 and 39) as to result in unsatisfactory speed regulation. Penstock velocities are seldom below 6.0 ft per second and in very high-head plants have been used as high as 20 ft per second. The typical velocity for medium-head plants is about 12 ft per second for maximum plant discharge.

When the general principles of economic design are applied to conduits, the following features must be taken into consideration.

Conduits in General. The annual cost must include the annual cost of all appurtenances that change with the size of the conduit.

High-line Conduits. The annual cost of high-line conduits must include the cost of grading, sills, cradles, trestles, and other important appurtenances and must also include the annual cost of the terminal regulator. If the terminal regulator is a surge tank, it will have some influence on the size of the conduit, because the size of the surge tank changes with the velocity in the conduit. If the terminal regulator is a pond created by a dam, the annual cost of the dam will vary, because the surges will be higher if the velocity in the conduit is higher. The internal pressure in closed conduits and the depth of water in open conduits are functions of the velocity in the conduits and of the size of the terminal regulator and should be considered in all problems of economic design.

The problem of economics is somewhat complicated for a closed conduit with a surge tank. Here there are three variables: the charges against the surge tank, the charges against the conduit, and the value of power lost. For a given size of surge tank, the most economic size of conduit can be determined only by successive trials.

The problem is not so difficult for open conduits with ponds for regulators, as the cost of the dam creating the pond usually varies inappreciably with the size of the conduit.

5. Air Valves in Pipelines. If pipelines, such as penstocks, which carry water under pressure, have vertical angles in them, then it may be necessary to install air-inlet or air-outlet valves in addition to the air inlet at the intake (Section 31, Chapter 27). A "summit" in the pipeline may require the installation of an air-outlet valve; otherwise air may accumulate at this summit until the pipeline becomes airbound. On the other hand, if a section of gently sloping pipe is succeeded by a section with a steeper slope downward, then an air-inlet valve may be required at the head of the steeper slope to prevent a collapse of the pipe due to vacuum.

It is, of course, desirable to avoid such vertical angles, and it is always better to have a pressure pipeline or penstock on a comparatively uniform slope. There are situations, especially in rough country, where the resulting sharp vertical angles cannot be avoided except at prohibitive expense, and it is in such situations that air valves should be used.

Figure 2 indicates the type of air-inlet valve manufactured by the Simplex Valve and Meter Company. The valve is kept closed by the buoyant effect of the water surrounding the float when the pipeline is full of water. When conditions arise which favor a vacuum in the pipeline, the buoyant effect is destroyed and the float drops from its seat, admitting air into the line. The valve will remain open regardless of the velocity or pressure of air until water again acts to buoy the float to its seat. For this reason the valve may also be used at summits of pipelines to release air.

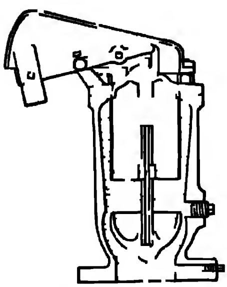

Fig 2 Cross-section of Simplex air-inlet valve (in the closed position) (Simplex Valve and Meter Co)

One of the large Coffin air-inlet valves used below the penstock valves at the Davis Bridge plant of the New England Power Company is shown in Fig 3. Its chief feature is the bronze float which floats up to its seat to close when the water in the penstock rises and drops to admit air when the water is drawn off from the penstock. The bottom of the float is perforated to prevent collapse under high pressure and also to reduce the shock of closure by permitting the water to rise in the float and compress the entrapped air. As shown in Fig 3, the opening of the valve is cushioned by the spring above the yoke. Valves of this type do not take care of small accumulations of air in the pipelines. For that purpose, if necessary, small auxiliary valves are installed

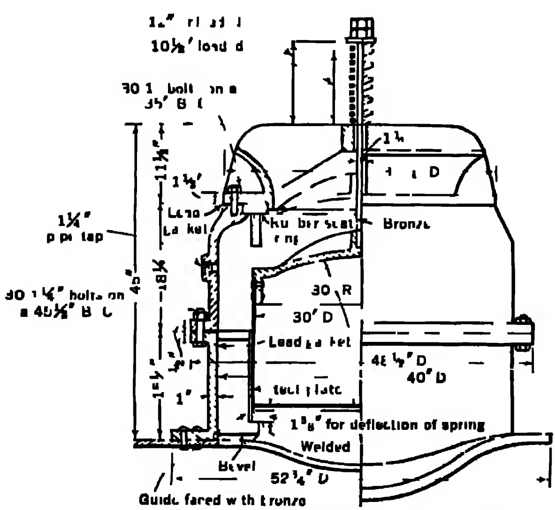

Fig 3 Air-inlet valve at Davis Bridge plant

The Simplex air-outlet valve of Fig 4 is used only to release accumulations of air in the pipelines, particularly at summits. The principle of operation is similar to that of the valve of Fig 2. When the pipeline is full of water

the float is buoyed and the valve is closed. As air accumulates, it replaces the water in the body of the valve and the float drops, allowing the en-

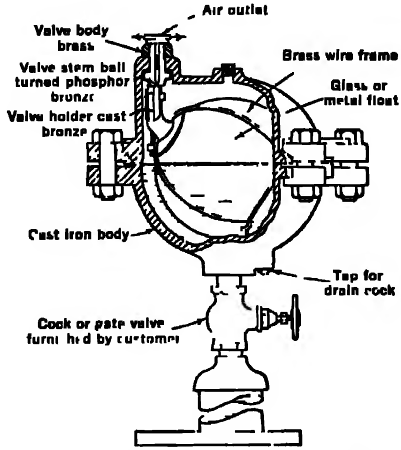

Fig. 4 Air-outlet valve (in the closed position). (Simplex Valve and Meter Co.)

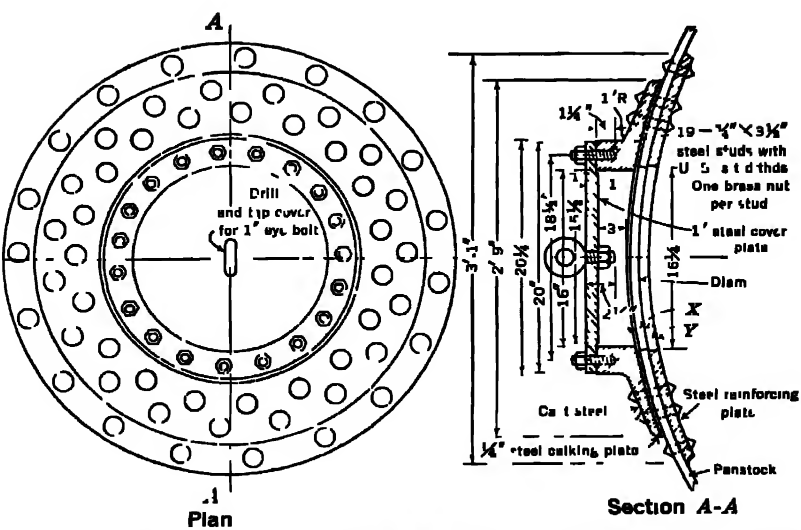

Fig. 5 Manhole for steel pipe Los Angeles Bureau of Power and Light. ("Penstocks" E E I Hydraulic Power Committee p. 36, 1936.)

trapped air to escape to the atmosphere. Water then returns to force the float upward and close the valve. The valve of Fig. 4 is designed to operate under working pressures up to 250 lb per sq in., but for special construction can be made sturdy enough to withstand pressures up to 800 lb per sq in.

Air valves of all types must be prevented from freezing. This feature is of the utmost importance. The valve of Fig. 3 was carefully inclosed in a separate compartment and freezing prevented by using a few electric space heaters in the compartment.

The required size of air inlet may be determined by methods discussed in Chapter 27, Section 31.

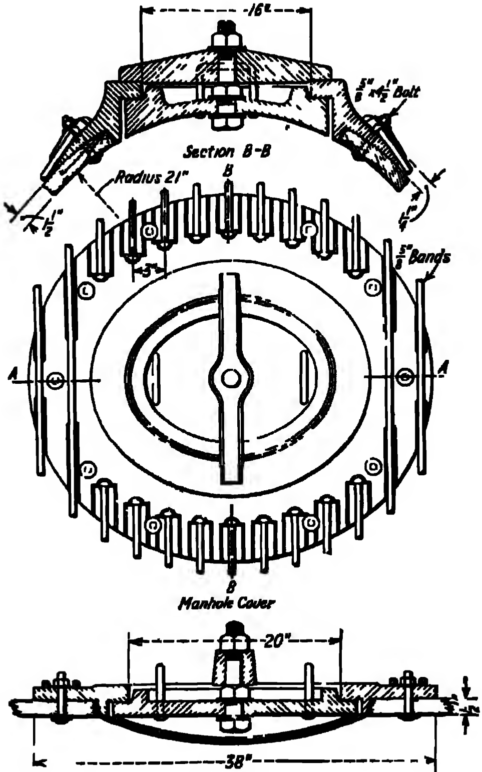

FIG. 6. Cast-iron manhole for 42-in. wood-stave pipe. (F. C. Kelsey, Consulting Engineer, N. Y. C.)

6. Manholes in Pipelines. A manhole should always be provided at each end of the pipe, and intermediate manholes should be located 1000 to 1500 ft apart. The manhole at the intake is usually provided by installing a ladder in the air inlet, and that at the turbine is frequently furnished with the steel spiral casing. Intermediate manholes of various types are available, most of which have been designed to reduce the disturbance to the flow in the pipe. Typical installations are indicated in Figs. 5, 6, and 7.

7. Conduit Valves. Gates and valves are used to permit unwatering of the conduit system for inspection and repairs. The various types of gates

described in Chapter 27 are always used at the conduit intake and serve to unwater the entire system. Valves can be used, but generally they are placed at strategic locations in the conduit to allow unwatering of sections of the system.

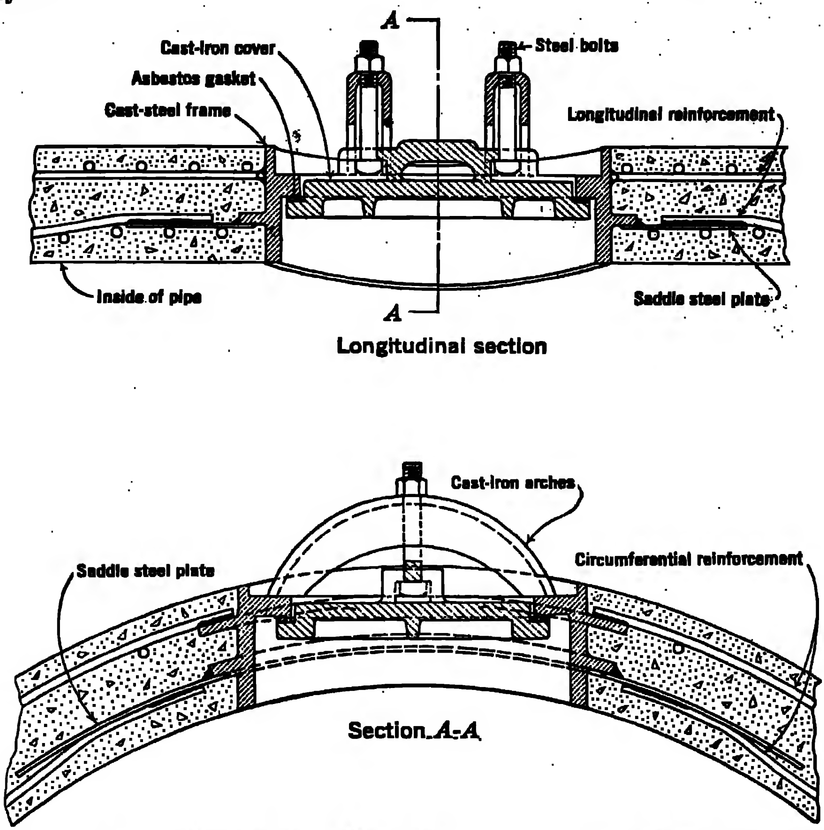

FIG. 7. Manhole for standard 14" x 16" opening in reinforced-concrete pipe. (Lock Joint Pipe Co.)

In any conduit, except a very short one, it is desirable to use a valve just upstream from the turbine, as in Fig. 11, Chapter 36, to reduce leakage when the unit is shut down or during off-peak periods. Thus, in case of injury to the turbine, this practice permits the prompt stopping of the column of water before it can do much damage. Needle valves, butterfly valves, and slide gates are used for this purpose.

The speed of operation of all conduit valves, if near the turbine, should be definitely limited to the speed of the turbine gates so that water-hammer pressure in the conduit will not exceed that caused by turbine-gate operation (see Chapter 34).

Butterfly valves have found considerable application as conduit valves in hydroelectric practice. They can be made very tight and have the advantage of requiring relatively small space outside the lines of the conduit. Butterfly valves are described in Section 17 of Chapter 27 and illustrated in Figs. 34, 35, 36, 37, 38, and 42 of the same chapter.

Needle valves are usually, and preferably, installed against the atmosphere at the outlet end of the conduit. Sometimes it may be desirable or necessary

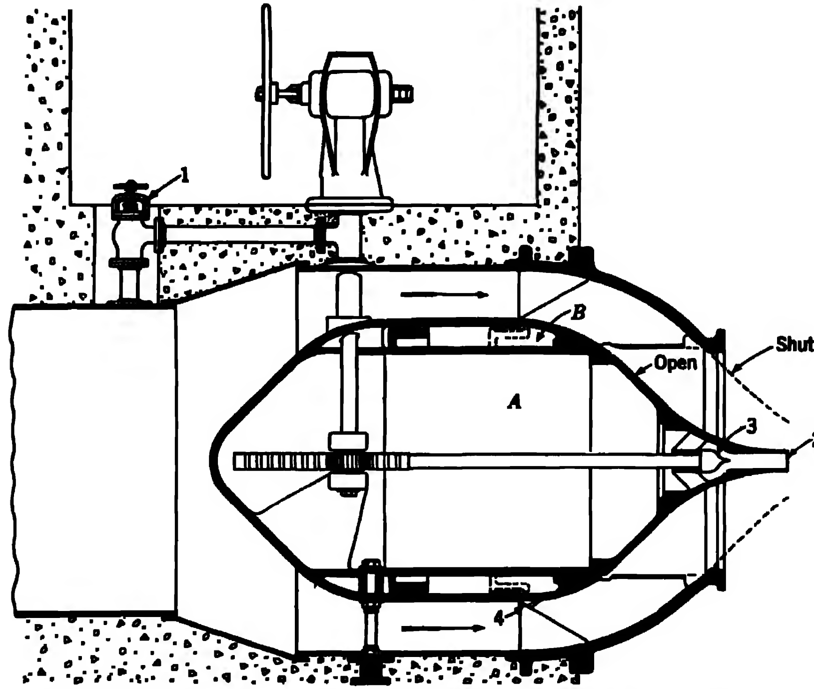

FIG. 8. Larner-Johnson valve. (I P Morris Division, Baldwin-Southwark Corp., Philadelphia, Pa.)

to install this type of valve in an intermediate position in the conduit. An expanding conduit section is then installed below the outlet and air is frequently admitted from an annular chamber surrounding the conduit close to the valve outlet. Needle valves may be damaged by freezing when exposed to low atmospheric temperatures, and their working parts may then require some maintenance attention. For these reasons guard gates are needed in the conduit and provisions must be made for draining the valve body and interior chambers.

An example of the *Larner-Johnson needle valve* is shown in Fig. 8. The flow passes through the outer annular passage. In operation, chambers *A* and *B* are connected to headwater. A pilot valve (3) in the head of the

plunger permits the release of water from chamber *A* and is actuated by the hand control. When the pilot valve is closed enough to restrict the flow through the nozzle (2) at the end of the plunger, the pressure in chamber *A* is raised and the plunger is advanced. Advance of the plunger tends to open the pilot valve, thus stabilizing the position of the plunger in accordance with the setting of the manual control. Similarly, opening of the pilot valve decreases the pressure in chamber *A* and retracts the plunger.

The *interior differential needle valve*, as shown in Fig. 23 of Chapter 26, is a more recent development of the needle valve by the U. S. Bureau of Reclamation. The distinguishing feature of this type of valve is an external control valve, known as the "paradox control," which controls the pressures in the interior chambers. The needle valve is divided into three chambers designated as *A*, *B*, and *C*. Chamber *A* is drained to atmospheric pressure. To close, conduit pressure is admitted to chamber *C*, and chamber *B* is drained to atmospheric. To open, these pressures are reversed. The paradox control is a piston valve arranged on the "follow-up valve" principle; i.e., the mechanism is arranged so that, with any manual setting, movement of the plunger in either direction will readjust the control port openings, so as to arrest or reverse the plunger movement. The plunger is in this manner kept close to the position determined by the manual setting of the control. When the manual control is moved, the plunger moves in the corresponding direction until its position is again stabilized by the readjustment of the control valve by the follow-up mechanism.

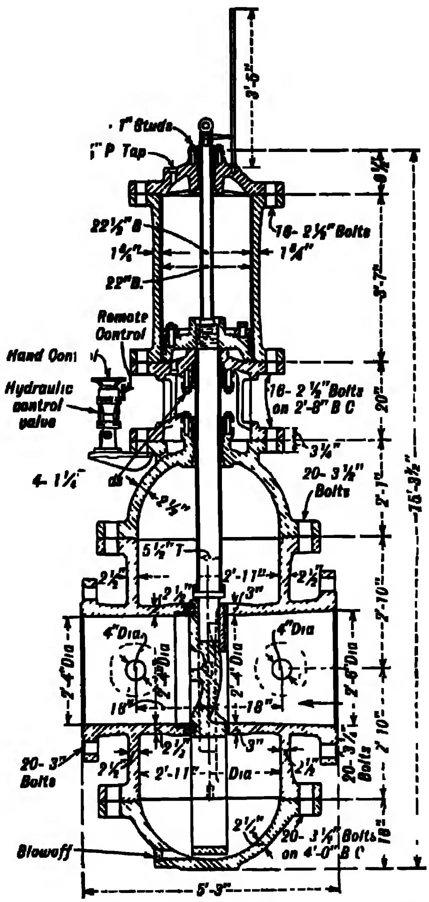

FIG. 9. Section through 28-in. gate valve for 2100-ft head. Note parallel seat of renewable bronze, slot-filling follow ring below plug, separate adjustable stuffing boxes on both valve and cylinder, bronze-protected rod, bronze-lined cylinder, indicating and lifting rod extending through cover, electrically operated control valve on side.

Gate valves have been used extensively in many plants but are becoming less general with the development of more efficient and less expensive types. Gate valves as large as 9 ft in diameter for 100-ft head and 28 in. in diameter for 2200-ft head have been built. Figure 9 shows a special 28-in. hydraulically operated gate valve built to operate under 2100-ft head. This valve was designed to be opened against full static pressure or to be closed against full-load velocity In order to meet these conditions the plug of the

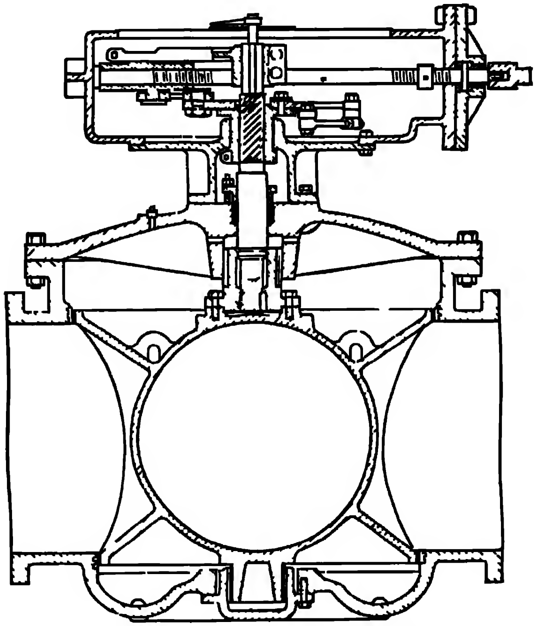

FIG. 10 Cone valve (Chapman Valve Co)

valve has a parallel seat and rests against this seat throughout its full travel. Valves for pipeline conditions need have a seat on only one end, as pressure conditions are never reversed, unless for test purposes

Gate valves have been practically superseded for diameters above 60 in. for medium-head conditions, although the small standard-type gate valves are still extensively used The most important installations of gate valves are for high-head conditions, probably from 800- to 2500-ft head and up to 48 in. in diameter. Valves for high-head conditions may be readily designed for penstock pressure operation by using a hydraulic cylinder direct-connected to the valve bonnet as in Fig 9 The inlet water should be strained, and the inside of the cylinder should be bronze-lined to prevent rusting.

High-head gate valves should be constructed of cast steel, and the parts subject to pressure are usually tested to about 50% above the maximum head which will be obtained at the plant. The standard type of gate valve is

suitable for low heads and small diameters, but the valves should be selected carefully and have parallel seats, as a tapered-seat valve may be broken off when closing against full-load velocity, because under these conditions the plug is not supported and the full force of the water must be carried from the stem.

Cone valves (sometimes called roto valves) are fairly recent developments and have had limited application in hydroelectric practice. However, they have been successfully used as check valves, stop valves, and control valves in penstocks and pipelines.

The principle of the cone valve, as shown in Fig. 10, is essentially that of a plugcock. The conical plug, with a circular orifice of the same diameter as the inlet and outlet connections, is seated in the valve casing. When in the open position the plug orifice lines up with the casing and the valve becomes a short pipe section with a free unobstructed passage. For closure the plug is unseated and rotated through 90 degrees to its seat in the closed position.

The advantages of this type of valve are the negligible loss in head, which actually amounts to a head loss for an equivalent length of pipe, and the smaller vault space due to compactness of the valve. However, like all valves in cold climates, it must be protected from freezing under certain conditions.

Cone valves are made in standard sizes from 6 in. to 60 in. for working pressures of 125 lb and 250 lb, but they can be specially designed for larger sizes. A typical installation is the Pinnacles development at Danville, Va., where three 24-in. cone valves are in service as shutoff valves in penstocks under a 660-ft head. Cone valves are manufactured by Chapman Valve Manufacturing Company, S. Morgan Smith Company, and Golden-Anderson Valve Specialty Company.

8. Blowoff Valves. Silt and sand are frequently deposited in deep depressions in closed conduits and must be removed through blowoff valves located in the bottom of the conduit at the lowest point of the profile.

The Howell-Bunger valve, shown in Fig. 24 of Chapter 26, is adaptable for this purpose. In this type of valve the jet is divergent and dissipated over a very large area, which reduces the effect of scour below the outlet.

The amount of deposits to be removed by a blowoff valve depends upon the rate of rise in grade of the conduit after passing the bottom of the depression, the velocity in the conduit, and the kind of material carried by the water. It is evident that, if the rate of rise is gradual and the velocity high, deposits will be negligible.

Blowoff valves may also be used to pass relatively small quantities of water into the channel downstream.

9. Excavation for Benches for Flumes and Pipes. Figure 11 shows a typical side-hill cut for a bench for a wood-stave pipe. The slope of the excavation varies, of course, with the nature of the materials. A width of 3 ft 0 in. for ditch D is not too great for a deep excavation if sloughing is likely to occur. For rock cuts, the ditch is eliminated and the distance D can be reduced to about 1 ft. The width C provided for sills depends on the

type of saddles but ordinarily will vary between $d + 1.0$ ft for very small pipe to $d + 2.5$ ft for large pipe. Distance C for steel pipe is, of course, less than that given above by the difference in thickness of shell; for flumes it varies with the type of structure. The outside berm, B, should be at least

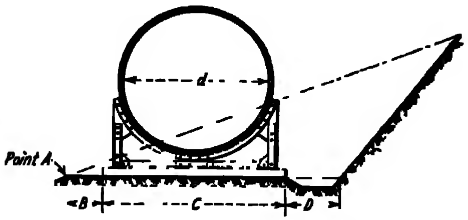

FIG. 11. Typical side-hill cut for a bench for wood-stave pipe.

2 ft for long, steep earth slopes on which the excavated material cannot be deposited. It can be reduced to zero if the excavated material can be deposited to the level of the bench.

It is not common practice to rest the sills partly on bench and partly on side-hill fill unless the fill has been deposited for a considerable period for full settlement.

10. Bibliography. This bibliography applies to conduits in general. References for the particular types of conduits are given in the bibliographies of Chapters 29, 30, 31, 32, and 33.

1. ARMIN SCHOKLITSCH, "The Power Conduit," *Hydraulic Structures*, translated by SAMUEL SHULITS, *A.S.M.E.*, Vol. 2, p. 764. 1937.

2. JULIAN HINDS, "Economic Size of Pressure Conduit," *Eng. News-Rec.*, Mar 25, 1937, p. 143.

3. JULIAN HINDS, "Economic Water Conduit Size," *Eng. News-Rec.*, Jan. 28 1937, p. 113.

4. J. M. SPAULDING, "Maintenance Practice for Penstocks and Flow Lines," *Electrical World*, Vol. 120, p. 163, Aug. 7, 1943.

CHAPTER 29

CANALS

1 General A general outline of the purpose and use of canals in hydroelectric developments and a discussion of those features of canals that are common to all types of conduits were presented in Chapter 28

2 Economics of Design For successful operation the size of the canal and its velocity for a given discharge may vary between wide limits, but

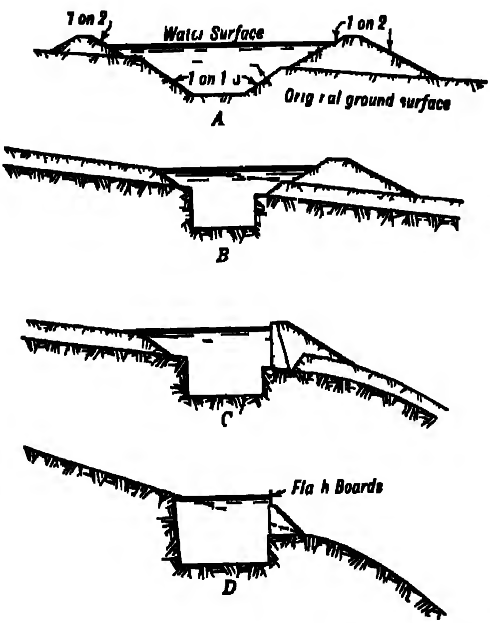

Fig 1 Several types of canals

for certain canals, particularly lined and rock canals there is one size only which will make for the greatest economy of design (Section 4 of Chapter 28)

3 Shape of Section Several types of canals are shown in Fig 1 There are of course, many other types for special conditions Canals in earth generally have a trapezoidal section canals in rock frequently have very nearly vertical sides, the slope of the sides being determined by the angle at which the rock breaks out most conveniently

Canal walls, as in Fig 1C are used only where sufficient room for an embankment is not available The canal wall shown in Fig 1D is, in reality,

a dam, as it has no backfill. Ice thrust between the rock face on one side and the concrete on the other is a source of danger and has caused several failures. For this reason, flashboards have been installed to bend with the expansion and contraction of the ice and to relieve the concrete from most of the thrust.

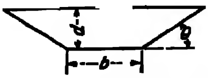

FIG. 2. Trapezoidal canal section.

With given area of cross-section and given slope, the form of cross-section that will give the greatest hydraulic radius will pass the greatest discharge and will therefore be (theoretically) the most economical form of cross-section. Thus, a semicircle would theoretically be the most economical form of cross-section but would be impracticable to excavate. It may be shown that for maximum hydraulic radius or minimum wetted perimeter for a trapezoidal section, as in Fig. 2,

$$d = \sqrt{\frac{A}{2} \frac{\sin \theta}{- \cos \theta}}$$

$$b = 2d \tan \frac{\theta}{2}$$

$$r = \frac{d}{2}$$

in which A = area of water cross-section;
θ = angle which the side slopes make with the horizontal;
P = wetted perimeter;
r = hydraulic radius = A/P;
d = depth of water.

For vertical sides, as in a rock cut, θ is 90 degrees and the bottom width for greatest hydraulic radius is twice the depth of water.

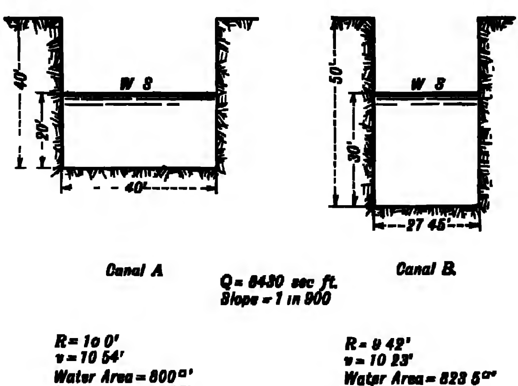

FIG. 3. Comparison of theoretical and practical canal cross-sections.

The foregoing theoretical considerations must be modified in practice, as other questions of economy usually will not allow the use of that section which has the maximum hydraulic radius. Thus, in a deep rock cut, the theoretical ratio of bottom width to depth of water must be reduced to decrease the amount of total excavation. Figure 3 shows two canals having equal friction loss per linear foot and equal discharge. Canal *A* is designed for the greatest hydraulic radius, while canal *B*, having a slightly larger water area, results in considerably less excavation. It will frequently be found, also, that the bottom width may more economically be made wide enough for power-shovel and track work, even at the expense of additional excavation.

In the same way it will be found that deep-cut and side-hill canals in earth will have a most economical section which does not correspond to that having the greatest hydraulic radius.

The section of canals is frequently influenced by the necessity for providing against ice troubles, as indicated in Section 10.

4. Side Slopes. In an unlined canal, the side slopes are determined by the slope at which the material will permanently stand under water. The nature of the material is a controlling factor. For example, loess may be satisfactorily excavated to a steep slope of 1 on ½ in the dry, but when in contact with water it will disintegrate into a fluid mass and assume a much flatter slope. The climate and the position of the water table also affect the stability of the side slopes.

Usually the slopes used in cut may safely be steeper than the slopes of the same material in fill. Thus in Fig. 1*A*, it is assumed that the material will stand in cut on a safe underwater slope of 1 on 1½. Then the underwater slope for the same material in embankment will probably be flatter than 1 on 2 for the same factor of safety against sloughing. When the material is sand and gravel, or a very sandy soil having little cohesion, this slope differential does not apply and the same slope may be used successfully for the water face of the embankment as in the cut.

Side slopes should be made as steep as practicable. For canals in rock the unlined slopes may be vertical or nearly vertical. Stiff clays may take slopes of 1 on 1 and give little trouble in most climates. Loose clays (unconsolidated) may take quite flat slopes. Sand slopes should seldom be steeper than 1 on 1½ and preferably 1 on 2. In dry climates on steeper slopes the sand sifts down to the bottom of the ditch.

A discussion of the side slopes required for lined canals is given in Section 11.

5. Choice between Lined and Unlined Canals. The factors that determine the choice between lined and unlined canals are:

(*a*) *The Velocity*. The velocity of the water in the canal may be great enough to erode the banks of an unlined canal seriously. (For eroding velocities see Section 7 and Fig. 4.)

The alternative to lining for protection against a high velocity is simply to increase the cross-section of the canal until the velocity is sufficiently reduced to prevent erosion. In open country this is usually the cheaper procedure.

(*b*) *Seepage Losses.* Where the ground-water table is below the bottom of the canal, and particularly in dry climates, there may be a material loss of water from the canal by seepage (Section 9). Many canals for both power and irrigation have been lined to prevent such loss. However, some water will escape even if the canal is lined. Canal linings are discussed in Section 11.

6. Minimum Permissible Velocities in Canals. (*a*) *Velocities to Prevent Sedimentation.* The velocity of water in a power canal should not fall low enough to permit the deposit of silt or débris. The minimum limiting velocity need not generally be given much consideration, because, as a rule, the intakes for power canals are located at the lower ends of ponds or reservoirs. That portion of the stream-borne débris which is fine enough to be readily deposited will be dropped in the pond near the point where the rapidly moving water of the stream comes in contact with the quiescent water of the pond. Hence, when the intake is from a pond or reservoir, material in suspension that has failed to settle out in the quiet water of the pond will not settle out to any extent in the comparatively rapidly moving water of the canal.

However, even in such a canal, if the water is laden with silt or clay and the plant is shut down, the canal, for the time being, becomes a settling basin and some of the silt and clay will be deposited. One might think that, when the plant was started up and the usual velocity re-established in the canal, the silt and clay thus deposited would be picked up again But this does not often happen, for the reason that the transporting velocity and the eroding velocity for any given material are frequently quite different. For sand and gravel the eroding and transporting velocities are not very different, but for clays and silt the margin is wide. Thus, if coarse sand or gravel is deposited in a canal, as soon as the velocity of the water has increased to just a little more than it was when the material was deposited, the sand or gravel will be picked up again and transported with the water. On the other hand, if clay or silt is deposited, the velocity of the water must increase greatly above the velocity at which the deposit took place before the material will be picked up again.

A mean velocity of 2 to 3 ft per second will generally be sufficient to prevent the deposit of silt.

(*b*) *Velocities to Prevent Plant Growth.* In some climates, the growth of aquatic plants and moss seriously affects the capacity of canals. It has been found that, when the temperature of the water is below 65 degrees Fahrenheit, algae and moss growths are not serious; nor do the growths take place to any extent in turbid or deep water. A mean velocity of not less than 2.5 ft

per second will generally prevent a growth that would seriously decrease the carrying capacity of the canal.

7. Maximum Permissible Velocities in Canals. *Velocities to Prevent Erosion.* The maximum permissible velocity in a canal is the greatest velocity that will erode the bottom and sides of the canal. Generally this limiting maximum velocity will determine the cross-section of an unlined canal, because an unlined canal that meets the criterion of economic conduit design would have a velocity high enough to erode the bottom and banks of the

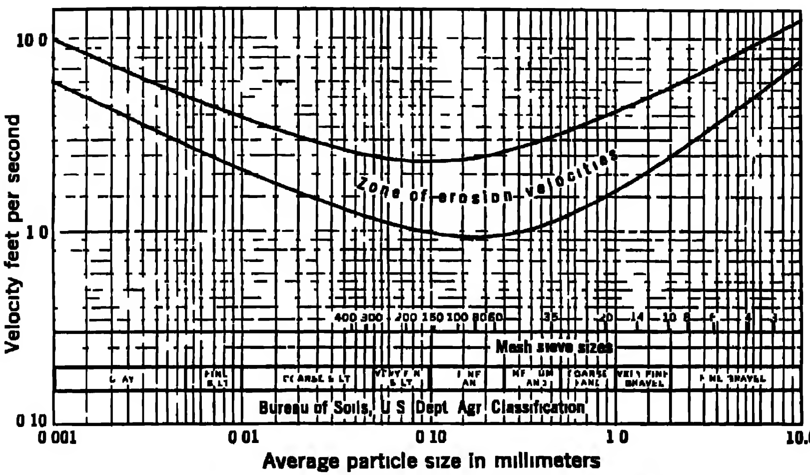

FIG 4 Erosion velocities for various particle sizes

canal Figure 4 indicates the zone of erosion velocities for various soil grain sizes from clay to fine gravel, on the basis of the recommended practice of various engineers

Unlined canals carrying water at velocities over 4 ft per second usually require riprap for bank protection Riprap details are discussed in Section 16 Chapter 22

When a canal is lined with concrete, no definite maximum limiting velocity can be set, provided that the water does not carry sand, gravel, or stones Velocities of 40 ft per second were successfully used in a concrete spillway chute of the Strawberry Valley irrigation project in Utah

In using very high velocities over a concrete lining, as is sometimes done in a spillway canal or channel, it should be remembered that there is a tendency for the rapidly moving water to pick up the blocks and move them out of position, because the pressure under the blocks sometimes exceeds that of the rapidly moving water just above. For this reason, when high velocities are used in such a channel, the concrete blocks are often made several feet thick, especially if the foundation on which the concrete blocks are laid consists of sand, gravel, or soil, instead of rock.

8. Design of Canal Embankments. For very short power canals, it is usually economical and desirable to make the elevation of the top of the canal well above the elevation of high-water surface in the pond. This obviates the necessity of using control works at the entrance to the canal and provides for full head on the turbines during periods of high water. With longer canals this practice becomes uneconomical, and control works with gates in them are located at the entrance of the canal, so that the water level in the canal can be controlled if there is an increase in the level of the water in the pond from which the canal takes its water. In such cases, the level of the water in the canal is also controlled by means of one or more spillways along the canal. In determining the proper freeboard to use for a canal, the surge, due to a sudden shutting down of the turbine gates, should be computed and allowed for (Section 15, Chapter 8), and due allowance should be made for possible errors in computing the depth of flow, as described in Section 11, Chapter 8.

Canal embankments are merely small earth dams, and the same principles which apply to the freeboard above highest surge, the width of top, the slopes, riprap, cutoffs, etc., in earth dams, are applicable to their design and construction. (See Chapters 20, 21, and 22.)

9. Seepage Losses in Canals. In canals in arid regions, seepage losses are a matter of great concern; but in power canals in most climates the losses due to seepage are rather insignificant. The seepage of water from embankments is discussed in Section 8 of Chapter 3 and Sections 14, 15, and 16 of Chapter 20.

A canal which is largely below the ground-water surface, other conditions being equal, will have practically no leakage. In dry climates ground water is often a valuable source of irrigation water. For this reason the seepage from a power canal which replenishes the ground-water supply is often economically justified.

A canal which has a great deal of embankment requires attention to make it sufficiently watertight. Such canals may often be effectively tightened by (*a*) compacting the sides and bottom, (*b*) an inexpensive treatment with bentonite, or (*c*) treatment with hydrated lime (Section 11).

10. Ice Trouble in Canals. Shallow canals in cold climates sometimes give a great deal of trouble in the winter because of ice, especially if the canal is long as well as shallow and the water flows at a relatively high velocity. Under such conditions the velocity is high enough to prevent the canal from freezing over to any great extent, with the result that frazil or anchor ice * often forms in considerable quantities, giving a great deal of trouble at the racks and sometimes at the unit itself, and frequently causing a shutdown of the plant. For these reasons, some other type of conduit is sometimes selected in preference to a canal for a project in a cold climate, even where a canal would apparently be the most economical.

* For definition see Section 6, Chapter 27.

However, ice troubles in canals can be obviated or at least reduced to an insignificant minimum by suitable design, construction, and operation. It is, therefore, not sound judgment to omit entirely the consideration of their use in a cold climate when a real saving could be effected thereby.

Narrow, deep canals are subject to fewer ice troubles than shallow, wide ones of the same capacity.

Other considerations, such as maximum economical loss of potential energy and maximum velocity for safety against erosion for the material through which the canal passes, will generally limit the maximum velocity in the canal cross-section to 3 or 4 ft per second. Under such conditions the velocity at night, when the load on most hydroelectric plants is at the minimum, will be but a small part of this, and the ice sheet will usually form before there is very much trouble from frazil or anchor ice. Generally speaking, the minimum velocity in the canal should be less than 1.5 ft per second in order to permit the formation of an ice sheet. With paper mills and chemical plants having a large night load, it is frequently desirable at the beginning of a cold snap to shut down or run light for a night or two in order to permit the ice sheet to form. Once an ice sheet has formed on a power canal, there is seldom any further trouble from frazil or anchor ice.

Assuming that the construction of the intake to the canal is such that ice cakes are not permitted to enter the canal, ice should rot out of a properly designed and constructed canal without causing much trouble. When the intake is located in a river that carries large quantities of cake ice, like the St. Lawrence, Niagara, or Susquehanna, the water is sometimes drawn from the bottom of the river at relatively low velocity. The direction of the current to the intake is often at approximately right angles to the surface current. The surface current of higher velocity then carries the cakes of ice past the intake. This arrangement is sometimes facilitated by excavating grooves in the bottom of the river to aid in conducting the lower stratum of water to the intake. Also, provision for a skidway to take care of cakes of ice should be made in the forebay near the intake to the penstocks or powerhouse.

11. Canal Linings. As mentioned in Section 5, canals are lined with concrete or other impervious material to control the seepage and to permit higher velocities. It should be noted that the excavation of the cross-section for a lined canal will usually be much less than the excavation of an unlined canal for the same project, for the following reasons:

(*a*) A lined section will give a much lower value of Kutter's n than an unlined section.

(*b*) A lined section will permit the use of a velocity giving an economical loss of head, instead of a velocity limited to the maximum for which the given material is safe against erosion.

(1) *Plain concrete slab linings* are frequently used for lining power canals. For this purpose the concrete is generally a 1:2:4 mix or even richer. One method is to form flat-slab panels 4 to 6 in. thick and from 10 to 20 ft long

which are placed against trimmed earth slopes. The advantage of dividing the lining into panels is to provide for the possibility of settlement or heaving of the material beneath. Thus the panels would move individually and eliminate the cracks that would occur in a monolithic lining under the same conditions. The division into panels also provides for expansion and contraction due to temperature changes. The panels are staggered to avoid a continuous transverse joint.

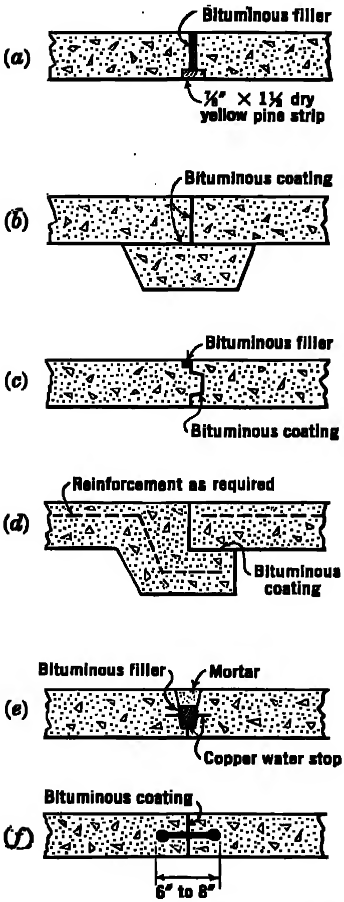

FIG. 5. Typical joints in concrete canal linings.

An effective manner of securing watertight joints between the panels is indicated in Fig. 5. The type of joint used may very well affect the cost of the lining. In type 5(a), the wooden strips make an effective and economical expansion joint. They are dry when put in place at the time of pouring of the concrete. They gradually take up water, swell, and become very tight. When the panel contracts, the contact between the concrete and the wood provides an effective water seal. The objection to the use of wooden strips is that they will decay if not kept continually wet. However, in a power canal, where the strips will usually be kept wet at all times, they may be expected to last indefinitely. Type 5(b) consists of a 4- to 6-in.-thick sill at the joint and level with the underside of the slabs. The surfaces of contact are coated with bituminous material. When the panels expand or contract, they slide on the sill and maintain watertightness. In this type of joint the sills can be installed in advance of the slab construction. In type 5(c) the keyed joint allows movement of the slabs but also keeps them in alinement. Bituminous filler is placed as shown to insure a watertight joint. Type 5(d) is similar to type 5(b) except that the sill is part of one slab. This type is used with reinforced-concrete slabs. The joint of type 5(e) requires a copper strip imbedded in the ends of both slabs in the form shown. The cavity is filled with a bituminous material and covered with a cement mortar to make a smooth finish. This refinement is undesirable in most cases. In locations where the concrete lining is always wet and in the dark, the yielding rubber seal, shown as type

5(*f*), is believed to be permanent. Under other conditions it should never be used. Many other variations of the types shown in Fig. 5 are possible to make up a watertight expansion joint.

(2) *Reinforced-concrete linings* are made in a thinner layer than the plain concrete slab linings. They are generally laid down monolithically without the use of expansion joints, or else these joints are spaced very much farther apart. It is claimed that a lining of this sort is tighter than one in which slabs are used The steel probably causes the concrete to open up in a large number of minute cracks instead of in a few large cracks as it would if no steel were used. The thickness of concrete varies from 1½ in. to 3 in. The steel reinforcement consists of wire cloth or expanded metal and is laid in position on the trimmed earth slopes and supported by means of chairs, small blocks of mortar, or strips of wood. The concrete is then poured, care being taken to retain the reinforcement in the concrete instead of allowing it to be pressed down against the earth by the weight of the fresh concrete. To insure this, it is often necessary during the pouring of the concrete to take a hook, pull up the reinforcement, and joggle it so that some of the concrete gets underneath.

When the slope of the concrete lining is 1 on 1 or flatter, it is better and more economical to place it without the use of forms, using a fairly stiff mixture and tamping it into place The surface should be floated and troweled A very smooth surface having a low value for Kutter's *n* can be obtained in this way. When the slopes are steeper than 1 on 1, forms are generally used; here a wetter mixture becomes necessary. The steel ratio generally used for such linings is about 0.3%. The concrete mixtures used vary from about 1:2:4 for the greater thicknesses of concrete to mortars without any stone or gravel for the 1½-in thicknesses. The mortars used for this purpose vary from 1 to 2½ to as lean as 1 to 5; a mortar as lean as 1 to 5 is not recommended but has been used. The real advantage of these thin reinforced-concrete linings is that they are cheaper than the concrete panels. An unreinforced-concrete lining as thin as this would soon disintegrate in most climates. The addition of the steel holds it together, and an effective lining is generally secured except under conditions where the frost action is especially severe.

(3) "*Gunite*" *concrete linings* have been placed in varying thicknesses from ½ to 2 in. or greater. After the slopes have been trimmed to grade and all loose earth or other material has been removed, steel mesh reinforcement is laid in place, being supported by means of small precast mortar blocks which have pieces of wire, for fastening to the reinforcement, cast in them. Chairs or other suitable means may also be used in lieu of the mortar blocks for supporting the reinforcement in place. The cement gun is placed in position near the work and the gunite is shot on. When the gunite is built up to the required thickness, all rebound material is removed and the surface is troweled smooth. This method gives a very strong and impervious surface.

(4) *Slopes for Concrete Linings.* The slopes used for the concrete lining at the sides of the canal vary from 1 on 2 to 1 on ½ for canals in earth. The selection of the slope used should be governed by the nature of the material on which the concrete lining must be placed. Thus a clayey material, which is likely to become partly saturated at times owing to the presence of ground water, would require that the concrete lining have a flat slope. On the other hand, if the material is sandy and free-draining but with enough cohesion to have a steep angle of repose, the concrete lining may be placed on a relatively steep slope. For canal linings in rock, the sides are frequently made nearly vertical.

Any lining built on a slope steeper than the angle of repose of the underlying material should be designed as a retaining wall.

(5) *Drainage of Concrete Linings.* Care should be taken that concrete linings are well drained if ground water is some distance above the bottom of the canal; otherwise the lining may be moved by the pressure of water on the underside when the canal is emptied. This is particularly likely to occur on the uphill side of side-hill canals. In very cold climates the lining may be moved by frost action when the canal is empty, even if the water pressure is insufficient to cause damage. Effective drainage may be obtained either by loose stone or by open tile drains under the canal floor which discharge at suitable intervals.

(6) *Miscellaneous materials for linings* such as stone, wood, clay, and stabilized earth have been used with some success. Stone laid in mortar is rarely employed. Wood linings are occasionally suitable for temporary service. Clay sometimes serves as a lining for the bottom of canals, but such material is readily eroded, particularly during the emptying and filling of the canal. Earth stabilized with hydraulic cement mixtures, asphaltic compounds, and earth materials is being developed as a low-cost lining, especially in irrigation work.

(7) *Economics of Linings.* The cost of the lining should be credited with (*a*) the sum of the capitalized annual value of the energy which would be lost through seepage if the canal were not lined; (*b*) the capitalized annual value of the energy saved by less friction loss in the lined canal; and (*c*) the difference in cost of excavation between the unlined and lined canal (plus or minus).

12. Canal Spillways. When side streams are allowed to discharge into long canals and when abnormal surges must be taken care of wi hout a material rise in the water surface of the canal, spillways along the canal are used. These are generally located near the powerhouse or in the forebay; but if the canal is long and there are a number of side streams discharging into it, it may be necessary to have several spillways distributed throughout its length. Such spillways are usually small overflow masonry dams. Where larger quantities of water must be passed through a limited length of spillway crest, siphon spillways, as described in Section 7, Chapter 26, and spillways surmounted by gates, as described in Section 10, Chapter 26, are used.

13. Side-stream Tributaries. When a canal is built along a side hill, the brooks and small streams discharging from the hills are sometimes permitted to discharge directly into the canal. Where the country is steep, such streams will usually have a high velocity during rains and will carry a large amount of silt and débris, which will often be deposited when the swiftly moving water of the brook impinges on the relatively slowly moving water of the canal. Under such conditions, the stream should be passed through a culvert or inverted siphon under the canal and not allowed to enter the canal at all, as the deposit of débris at the mouth of the stream in the canal will either have to be periodically removed or will form a submerged dam in the canal which will seriously affect the carrying capacity of the canal.

Where the canal embankment crosses relatively long and wide stream valleys, the same objection does not exist, for a considerable pond is thus created and the débris will be deposited near the upper end of the pond where the stream enters it. Eventually, of course, the pond may be silted up, and this factor should also be given consideration. This may not be important, however, because of the length of time usually required for the pond to silt up to such an extent that deposits will begin to take place in the canal. Moreover, as the depth of water in such instances is usually very great, scouring sluices can be arranged in the base or sides of the canal to remove deposits.

Large ponds formed by side streams are quite desirable, particularly at the lower end of the canal, because of their ability to act as regulators to supply the peak demand, allowing the canal above them to be designed for less than the peak flow.

14. Sand Traps and Desilting Basins. When the intake for the canal is at a diversion dam in a rapidly flowing river, the velocity of the water in the canal may, at times, be very much less than that in the river, and under such conditions deposits in the canal are likely to occur. Even if the velocity in the canal is high enough to keep the sand in suspension, the resulting wear on the sealing rings of the turbines may be serious, increasing clearances and decreasing efficiencies. To obviate this difficulty, the use of sand traps and settling basins has been introduced. The problem is much the same as that presented in the design of settling basins and coagulation basins in filtration work, except that the permissible velocities are much higher. When there is a terminal pond or a large forebay, the velocity through either is so low that sand of a size to injure the turbines is deposited. If the forebay is utilized for this purpose, special means for removing the sand, akin to those described below, are frequently advisable.

In the design of settling basins, silt particles less than 0.05 mm in the water diverted through the turbines are considered too fine to cause appreciable wear on turbine parts. The length of settling basins depends upon the settling velocity of the silt particles. The settling velocity, which depends upon the size, shape, and specific gravity of the particles, can be determined by experiments on the material in question. On the basis of the data for the

design of the desilting works for the All-American Canal, discussed later, the minimum length of settling basin, l, may be taken as $l = 21.5d$, where d is the net depth of the basin above the silt deposit just before the silt is removed from the basin.

The velocity in the channel of 0.22 ft per second, required to settle out particles 0.05 mm and larger, can be obtained with a wide shallow basin as indicated in Fig. 7, or a narrow deep one. There must be sufficient space

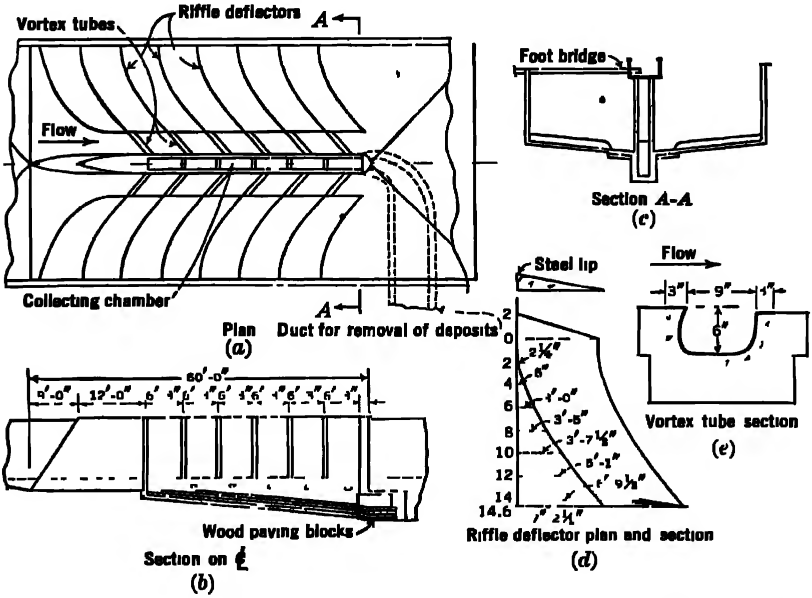

FIG. 6 The "riffle deflector-vortex tube" sand trap (H A Burt *Sand Traps in Open and Closed Conduits*, unpublished)

below the net depth, d, to receive the silt so deposited; this space depends upon the amount of silt expected, the interval of time desired between periods of removal, and the method of removal. A spare basin must be provided if continuous flow is desired.

Great care must be used in distributing the velocity throughout the cross-section of the settling basin so that all parts of the cross-section have practically the same velocity. Otherwise, crosscurrents will be produced and will prevent the deposit of a large part of the sand This even distribution of velocity may be accomplished by means of a baffle wall or submerged weir, as is common practice in settling basins for filtration plants But such devices require a loss of head of several inches, which is, of course, very objectionable in a power canal.

Figure 6 shows a sand trap developed by R. L. Parshall [9] for the Soil Conservation Service of the United States Department of Agriculture, in co-

operation with the Colorado Agricultural Experiment Station at Fort Collins, Colo. This "riffle deflector-vortex tube" type of sand trap is designed for the removal of material of varying degree of fineness. As indicated in Fig. 6(*a*), the riffle deflectors, shown in detail in Fig. 6(*d*), move the bed load toward the vortex tubes, shown in detail in Fig. 6(*c*). The vortex tubes trap the material and discharge it into the middle chamber where it is drawn off through a rectangular duct. This type of sand trap has shown practical application, in both small and large channels with a discharge ranging up to 1000 cu ft per second, for handling material ranging from very fine sand to small cobblestone, and has indicated an efficiency as high as 90% in the removal of sand. For channels 8 to 16 ft wide, the riffle deflectors should be on one side, with the vortex tubes and outlets on the opposite side.

The expense of removing silt from the All-American Canal formerly approximated a million dollars a year. The Bureau of Reclamation was determined to remedy this almost intolerable expense and designed and constructed the desilting basin at Imperial Dam [8], the canal intake from the Colorado River. As indicated in Fig. 7, the desilting works consists of a series of six settling basins arranged in pairs with a separate channel feeding each pair directly from the headworks at the dam. Each basin is approximately 769 ft long, 269 ft wide, with an average depth of 12.5 ft, and is set at an angle of 60 degrees with the main channel. The influent channel for each pair of settling basins has a diminishing cross-section with specially designed vertical slots in the channel walls to give an even distribution of the velocity through the basins. The water flows across each basin and over a weir, where it is collected in an effluent channel and led into the canal. Each effluent channel is also designed to operate as a by-pass channel if it should be necessary to take a settling basin out of service.

The desilting works at Imperial Dam were designed for a total flow of 12,000 sec-ft. A maximum velocity of 0.22 ft per second across the settling basin and a detention period of 21 minutes is required for the removal of all silt down to 0.05-mm particle size. Dorr rotary-type scrapers, 125 ft in diameter, scrape the deposited silt into collecting trenches at the pedestals. With a 6-ft hydrostatic head, the sludge, a solution of 10% silt concentration, is sluiced from the pedestals to the main collector pipes. Each basin has its own concrete sludge-pipe gallery or tunnel which carries a main collector pipe and directs the flow of sludge into the river below the dam.

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CHAPTER 30

FLUMES

1. General. The use of flumes in connection with hydroelectric plants was, in 1948, of minor importance in the United States as compared to 40 years before; but in newly developed countries where first cost may be of major importance they are worthy of consideration.

Those features of flumes which are common to all types of conduits are discussed in Chapter 28.

2. Types of Flumes. A flume is simply a long trough of wood, concrete, or steel, supported on or above the ground surface, and used to convey water. Steep, rocky hillsides, heavily wooded and with light soil cover, together with a necessity for low first cost, would favor the selection of a wooden flume made from lumber sawed on the ground. A similar topography, but without the timber and with the presence of suitable concrete materials, would favor the use of a reinforced-concrete flume, or, if suitable concrete materials were not present, the use of steel flumes.

3. Economic Design. For successful operation, the size of a flume for a given discharge may vary between wide limits; but there is usually one size which will make for greatest economy of design (Section 4, Chapter 28). In flumes there is no practical limit to the velocity, as it is always determined by the principles of economic design unless the conditions are unusual. Usual velocities are 8 to 10 ft per second.

4. Freeboard. The freeboard, or elevation of the sides of the flume above calculated high-water surface, is almost invariably used to provide a factor of safety to cover inaccuracies in the determination of the water surface. The freeboard should also be high enough to retain the surge due to a sudden shutting down of the turbine (Section 15, Chapter 8), unless the overtopping of the sides under such conditions would cause no damage.

For very short flumes it is usually economical and desirable to make the sides of the flume at least as high as ordinary high-water surface in the pond, to insure full head on the turbines when the water rises. If the flume cannot be overtopped without damage, the sides must be carried throughout to or above highest water surface in the pond, or control gates must be installed at the intake to limit the discharge into the flume.

Such control gates are invariably used for very long flumes, and the head due to high water is sacrificed for reduction in first cost of the flume. Under such conditions, spillways should be provided at convenient places to dis-

charge the excess flow in case the control gates are not kept in exact adjustment with the head on the intake.

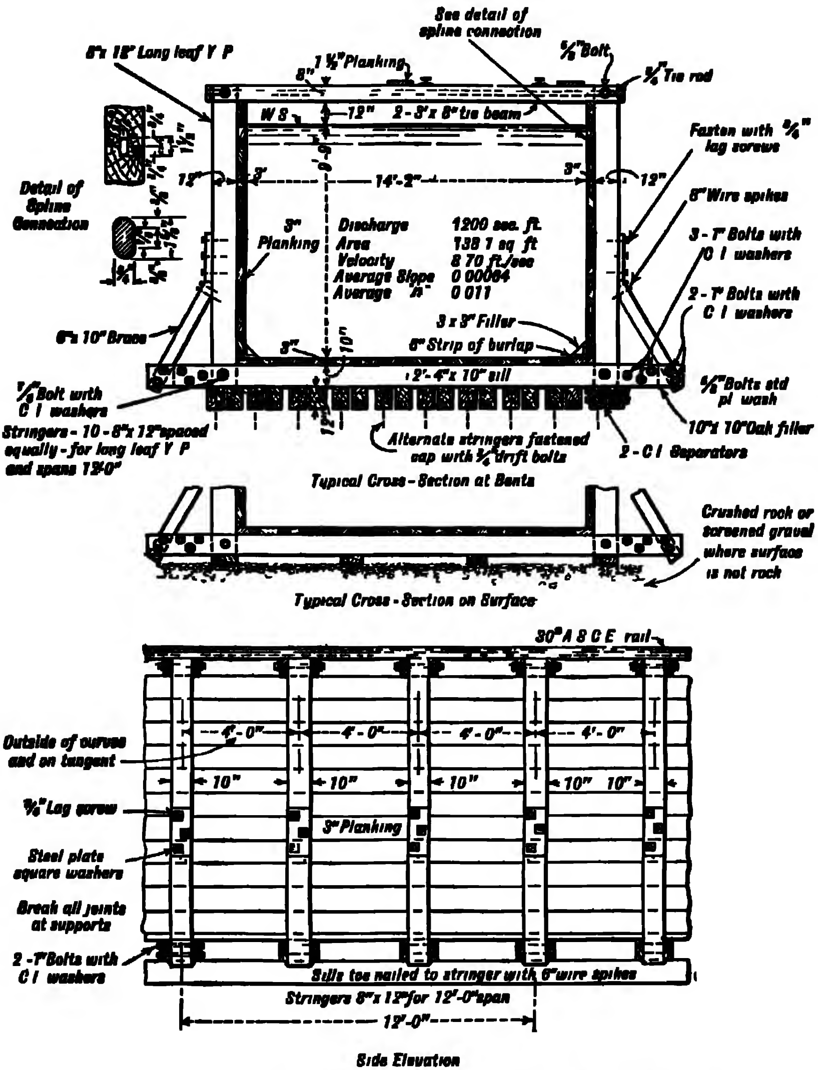


FIG. 1. Flume of Ocoee No 2 development, Tennessee Power Co.

5. Wooden Flumes. Untreated wooden flumes are used seldom and only where low first cost is of prime importance The life of wooden flumes is determined by the same conditions as given for wood-stave pipe in Section 13 of Chapter 32 The ultimate life of this type of flume, however, is about

10 to 15 years less than for a wood-stave pipe under similar conditions, because wood-stave pipe, being under some pressure, is more nearly completely saturated at all times. Creosoting increases the life of wooden flumes by 50 to 100%.

Two types of wooden flumes are indicated in Figs. 1 and 2. The lining should be of good-quality timber, free from warps and knots, and should have a thickness of 1½ in. or more, even for small flumes, since thinner boards tend to warp or crack. Supporting trestles, if used, should be made sufficiently

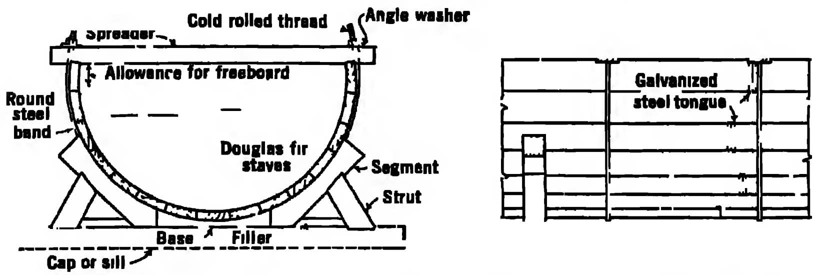


FIG. 2. Semicircular wood-stave flume. (Federal Pipe and Tank Co.)

strong to resist overturning by wind action in addition to carrying the water load. The trestle bents are preferably supported on concrete piers.

A rectangular wooden flume, as indicated in Fig. 1, consists of vertical studs to which the side lagging is nailed, and sills, to which the floor lagging is nailed. Generally the sills extend beyond the studs and are dapped at the ends to take the thrust from inclined braces supporting the studs. The studs are usually also secured by being tied across the top of the box, as shown in Fig. 1, either by a rod or a timber bolted to the studs. The lagging for the lining, as indicated in Fig. 3, may be (*a*) butt-end calked with oakum and pitch, (*b*) ship-

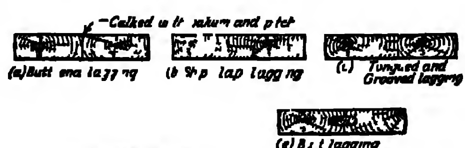


FIG. 3. Various types of lagging for timber flumes.

lap, (*c*) tongue-and-groove, (*d*) double lining, or (*e*) butt-end with splines. The butt lagging, with splines either of good-quality timber or of galvanized steel, when properly constructed, is the most desirable. Watertightness at the end joints is sometimes secured with battens or splines or by calking with oakum and pitch.

If the flume is on a bench cut in a hillside, the sills may rest directly on the ground or on blocking. Broken stone or coarse gravel under the sills, to facilitate drainage, prolongs their life. If the flume is on a trestle, the sills

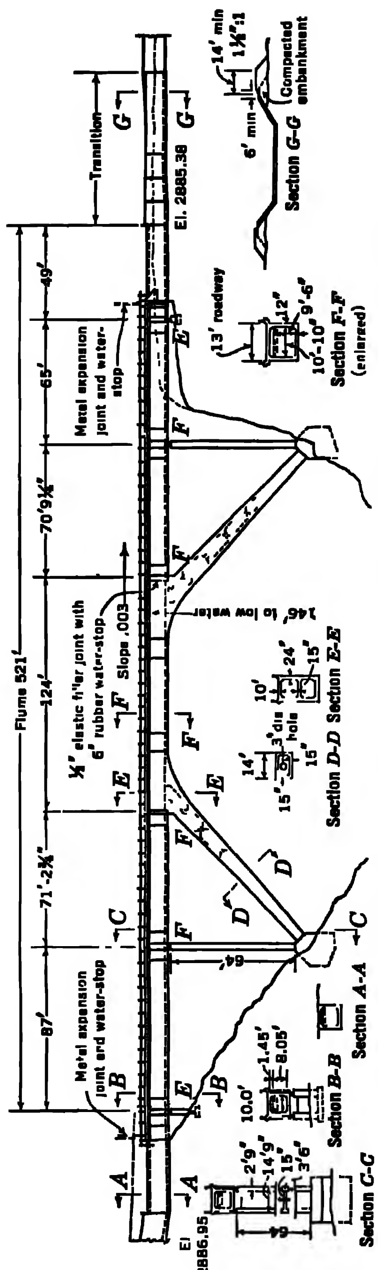


FIG 4. Reinforced-concrete rigid frame flume. (*Eng. News-Rec.*, p. 87, Oct. 3, 1946.)

rest on longitudinal stringers, as shown in Fig. 1, which carry the load of the flume from trestle bent to trestle bent. The span between bents is usually from 12 to 18 ft.

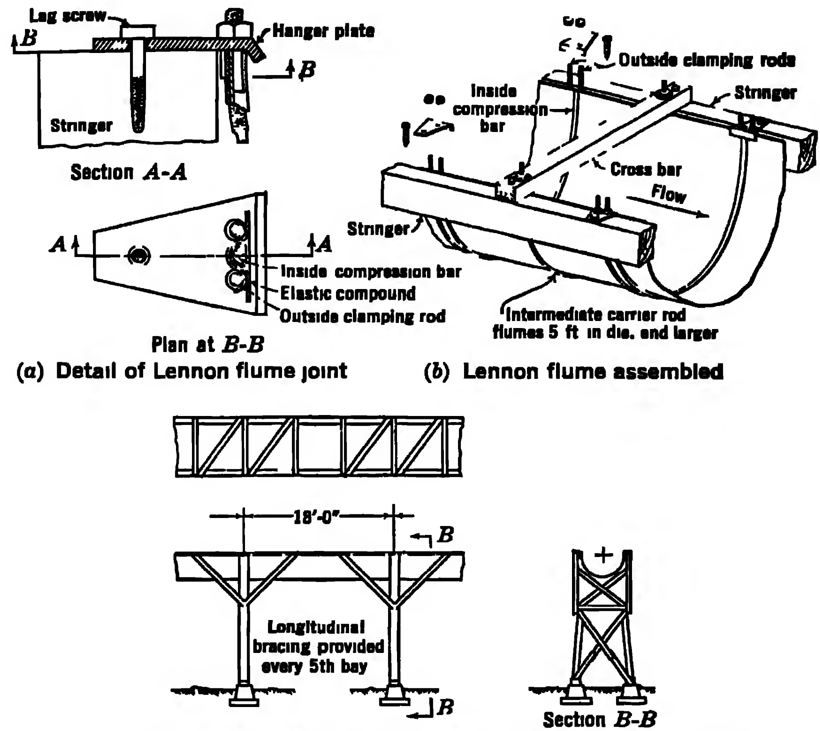
The details of the semicircular wood-stave flume, shown in Fig. 2, and the method of construction are similar to those of wood-stave pipelines (see Chapter 32). All staves and cradles should be creosoted to promote long life. As indicated in Fig. 2, there is a horizontal yoke, or spreader, to which the steel rods that confine the staves are attached. The spacing of the cradles is usually 8 ft center to center.

6. Reinforced-concrete Flumes. Reinforced-concrete flumes are almost invariably the most expensive type but also the most permanent and satisfactory. Figure 4 shows a flume designed as a rigid-frame structure. Reinforced-concrete flumes are more frequently built as bench flumes, thereby saving on the cost of the trestle.

7. Metal Flumes. Metal flumes may be constructed either as the rectangular or the semicircular type. In the rectangular type, the sides are usually designed as plate girders to carry the load of the flume and its contents and are sometimes tied across the top with steel rods or angles. The semicircular type is widely preferred to the rectangular type because it is cheap, light, easy to construct, and reasonably watertight.

Several patented types of semicircular steel flumes are on the market. They are composed of sheets of metal and are made in sizes running from about 1 ft to about 20 ft in diameter.

Semicircular metal flumes, as indicated in Fig. 5, are supported on either steel or wood trestles which are also designed to sustain wind load. The popular Lennon type of metal flume, Fig. 5a and b, is described as follows: Both edges of each sheet of metal are crimped to form a groove. In assembling, the sheets are placed so that the grooves of adjacent sections overlap.



(a) Detail of Lennon flume joint (b) Lennon flume assembled

(c) Type of steel substructure for metal flumes of medium height

FIG. 5. Details of a semicircular metal flume. (Armco Drainage and Metal Products, *Handbook of Water Control*, 1946.)

A rod or light curved channel (inside compression bar) then fits into the groove on the inside of the flume, and two outside clamping rods fit around the flume on the outside of the groove. The outside rods are threaded at their ends and pass through a hanger plate on the stringer. By screwing up the nuts on the outside clamping rods, the sheets are pressed tightly together between the inside compression bar, which is seated against the hanger plate, and these outside rods. The recess in the joint is filled with an elastic compound to make a smooth interior surface.

Metal flumes are sometimes eroded by water laden with débris. The erosion is usually controlled by treating the interior surface with coal-tar enamels every 3 or 4 years.

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CHAPTER 31

STEEL PIPE

Carl H. Scheman and Harry C. Boardman,† collaborators*

1. General. A general outline of the purpose and use of pipes in hydroelectric developments and a discussion of those features of pipes which are common to all types of conduits are presented in Chapter 28. The pipe between the forebay and the surge tank is commonly called the "high line conduit," and that between the surge tank and the turbines is called the "penstock."

2. Types of Pipe. The two main types of steel pipe used in hydroelectric developments differ only in the kind of joints, i.e., "riveted pipe" and "welded pipe." In the older installations, before 1930, riveted pipe was much more common. By 1947, most installations were of welded construction. Examples of such installations are: the U. S. Bureau of Reclamation development at Hoover Dam in Arizona-Nevada; the Aluminum Company developments at Shipshaw in Quebec and at Nantahala in North Carolina; the Southern California Edison Company developments at Big Creek in California and in Tennessee; and the Tennessee Valley Authority developments at Fontana Dam and Apalachia Dam.

In the main, a well-designed and fabricated welded job is the lightest job. Good welded construction produces a finished job of lower hydraulic loss than a riveted pipe of equal size.

3. Loading. Loading influences particularly the thickness of plate and the longitudinal joints. Loadings comprise the pressure, both static and dynamic, which includes the weight of water in the pipe, water hammer, and the weight of the pipe itself. The maximum internal pressure for which closed conduits must be designed consists of the maximum static water pressure, plus the maximum water-hammer pressure due to sudden reduction in velocity, plus the maximum excess pressure due to fluctuations in the water surface in the surge tank. The theory of water hammer is treated in Chapter 34 and the excess pressure in the conduit due to surges in the surge tank in Section 16 therein and in Section 7 of Chapter 35.

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The equation for the thickness of thin cylindrical shells under internal water pressure may be written as

$$t = \frac{2.6hd}{SE} \qquad [1]$$

where h = the pressure at the center line of the conduit, due to all estimated loadings, in feet of water;
d = the diameter of the conduit, in feet;
S = the permissible stress in steel, in pounds per square inch;
E = the longitudinal joint efficiency expressed as a decimal;
t = the required steel plate thickness, in inches.

Stresses due to partially filled pipes tending towards distortion and to reactions at anchorages, supporting saddles, and rings create additional problems that must be solved.

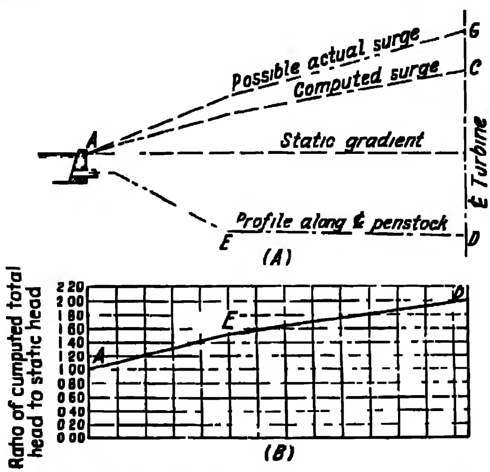


FIG. 1. Method of providing a margin for inaccuracies in surge-pressure computations.

The service stress in the steel members of any structure should never exceed the elastic limit. The unit stress used in designing is chosen less than the elastic limit to provide a margin against errors in analysis and estimated loadings, and defects in materials and construction.

The usual practice of adopting a low unit stress to compensate for inaccuracies in the loading assumption is not applicable to conduits for hydroelectric developments, since the percentage of surge, or load above static, is not constant along the conduit. In Fig. 1A, the line AC is the computed surge gradient (water hammer or surge in pipe leading to a surge tank). The ratio of the computed total head to static head is indicated in Fig. 1B, where it is seen that the computed total head is 200% of static at D and only

150% at *E*. Consequently, a constant nominal factor of safety to allow for errors in computed load would be erroneous, since an error of 50% in the computed surge would result in an increase in stress of 25% at *D* and only 16.7% at *E*.

It is therefore recommended that the computed surge pressure be increased by a constant percentage, as in *AG*, Fig. 1*A*, to allow a margin for inaccuracies in its computation, and that the working unit stress in the conduit, based on gradient *AG*, be lower than the elastic limit only by an amount sufficient to allow for defects in material and fabrication. A constant thickness, usually ¼ in., may be added to the plates of steel pipes to allow for corrosion.

The constant percentage to be added to the computed surge in order to obtain gradient *AG* varies with the existing conditions. The pressure due to surges in a surge tank is practically correct. Pressures in pipelines due to water-hammer pressure required to accelerate the water in the riser pipe of a surge tank and to water-hammer pressures in penstocks cannot be determined exactly. Recommended percentages are given in Section 17, Chapter 34.

TABLE 1

PHYSICAL PROPERTIES OF MATERIALS FOR STEEL PIPE

Item	Serial Designation of American Society for Testing Materials	Specifications	Grade	Ultimate Tensile Strength, min, lb per sq in	Yield Point*: Percentage of Tensile Strength	Yield Point*: Minimum Not Less Than, lb per sq in
Welded pipe	A283 46T	Tentative specifications for low and intermediate tensile strength carbon steel plates of structural quality	A	45 000 to 55 000	50	24 000
			B	50 000 to 60 000	50	27 000
			C	55 000 to 65 000	50	30 000
			D	60 000 to 72 000	50	33 000
Riveted steel pipe	A30 46	Standard specifications for boiler and firebox steel for locomotives	*Flange Steel* Flange	55 000 to 65 000	50	27 500
			A Firebox	55 000 to 65 000	50	27 500
			B Firebox	48 000 to 58 000	50	24 000
Rivets	A31 40	Standard specifications for boiler rivet steel and rivets	A	45 000 to 55 000	50	22 500
			B	58 000 to 68 000	50	32 000
Steel castings	A27 46T	Tentative specifications for mild to medium strength carbon steel castings for general application	U 60-30	60 000		30 000
			60-30	60 000		30 000
			65-30	65 000		30 000
			65-35	65 000		35 000
			70-36	70 000		36 000

* The yield point given in this table is the "commercial elastic limit."

TABLE 2

ECONOMICAL DIAMETER OF PIPE

Head = 100 ft; length = 100 ft.
Q = 200 cu ft per second = average discharge.
e = 0.80 = over-all average efficiency of equipment.
H_f = line 4 = loss of head in feet at Q.
C = 2.0 = factor to correct loss of head at Q to average conditions of loss of head.
T = time = 8760 hr = 1 year.

1. Diameter of pipe in feet	5.5	6.0	6.5	7.0	7.5	8.0
2. Construction costs	$5,300	$5,900	$6,510	$7,100	$7,710	$8,300
3. Annual operating costs	135	150	165	180	195	211
4. Loss of head in feet	0.260	0 155	0 103	0.072	0.052	0 038
5. Annual lost energy in kw-hr	62,000	37,000	24,600	17,100	12,400	9,000
6. Value of energy lost at 10 mills per year per kw-hr	$620	$370	$246	$171	$124	$90
7. Increments of increased gross return for each 0 5 ft increase in diameter (from line 6)		$250	$124	$75	$47	$34
8. Increment of increased operation charge for same conditions (from line 3)		$15	$15	$15	$15	$16
9. Increment of increased net return for same conditions (line 7 minus line 8)		$235	$109	$60	$32	$18
10. Increment of construction costs for same conditions (from line 2)		$600	$610	$590	$610	$590
11. Per cent net return on increment investment (line 9 ÷ line 10)		39.2	17.9 *	10.2	5.3	3.0
12. Average velocities in pipeline	8.4	7.1	6.0	5.2	4.5	4.0

* Approximate most economical size of pipe for given assumptions.

Line 2. The cost is assumed for pipe with 100-ft head. Figures are only for illustration.

Line 3. Annual operating charges include operation, maintenance, repairs, taxes, depreciation, and all other annual charges affected by the diameter.

Line 4. Loss of head is the loss of head for the average discharge using tables for fairly smooth pipes.

Line 5. Lost energy is computed from formula: kw-hr = $QCH_feT/11.8$. Substituting in this formula, line 5 = 238,000 times line 4.

Line 6. For the purpose of this example 10 mills per year for kw-hr was used as the value of the energy. The value of the energy will vary greatly between plants and will vary at any one plant depending upon whether the energy is base or peak. Base energy may be worth only 4 mills while peak energy may be worth 12 mills.

Lines 7 to 11 are explained in the table.

The 17.9% return on the investment in line 11, to increase the diameter from 6.0 to 6.5 ft, is the average rate of return for all infinitesimal increase between 6.0 and 6.5 ft and is approximately the actual rate of return for a small increase at 6.25 ft.

Inasmuch as it is assumed that, under 1948 conditions, a return of 15% is desirable in order to justify an incremental capital expenditure, our conclusion is that the pipe selected should be about 6.5 ft in diameter.

Where the units are of large water capacity, there is usually one pipeline per unit.

The recommended working stress for use in steel-pipe design is 50% of yield strength as given in Table 1. This table gives the specifications of the American Society for Testing Materials, which are usually applied to the steel for pipe. The lower-strength steels are recommended because of their greater ductility and workability.

The problem of design depends somewhat on whether the weight of the pipe and its contents is transmitted to the piers through saddles, which cover only part of the circumference, or through ring girders, which encircle the pipe. Modern design, discussed in Section 7, uses ring girders.

4. Determination of Economic Diameter. For successful operation, the size of the pipe, for a given discharge, may vary between wide limits, but here is usually one size that will make for the greatest economy in design. (See also Section 4, Chapter 28.) Penstock velocities may range from 6 to 20 ft per second. The typical velocity for medium-head plants is about 12 ft per second for maximum plant discharge. (See also Section 19, Chapter 38.) The shape of the load curve will have considerable influence in determining the maximum economic velocity. Thus, if the peak is of very short duration, it may be economical to use a relatively high velocity during this period.

In order to determine the most economical size of pipe it is desirable to make several cut-and-try calculations taking different pipe sizes and computing incremental costs and energy losses, applying the correct value of the energy, and finally obtaining a percentage return for that particular increment of pipe size. A particular design is most economical when an infinitely small additional investment to increase the energy output will give a percentage return exactly equal to the minimum desired net return on the money invested. Table 2 illustrates the method of incremental values.

5. Riveted Steel Pipe. The efficiency of various types of riveted joints varies with the number of rows of rivets and may range between 55 and 95%. The most economical type of joint for given conditions depends upon the head and size of the pipe. The most efficient joint is also the most ex-

pensive joint, and whether an increase in the cost of the joint to reduce the thickness of plate is justifiable is a problem in relative economy.

Some engineers add about ⅛ in. to the calculated thickness of plate to allow for possible corrosion, but this is questionable practice except for extremely thin plates or perhaps for buried pipe. Exposed pipes are easily taken care of on the outside, and experience has shown that pipes constantly full of water corrode very slowly on the inside even if the paint is allowed to deteriorate. A minimum thickness of plate of ⅜ in. is common practice, although a number of pipes have been installed with a minimum thickness of ¼ in.

A minimum thickness is required to give sufficient stiffness at and between the saddles or ring girders, and to provide for a reasonable amount of corrosion of the surface without too large a percentage reduction in thickness. Sometimes, riveting or welding circumferential angles, or other members, to the pipe provides stiffeners to increase its stability.

The allowable stresses for power-driven rivets (both shop and field) are: shear, 15,000 lb per sq in.; bearing, single shear, 32,000 lb per sq in., double shear, 40,000 lb per sq in.

Report D-15 of the Hydraulic Power Committee of the Edison Electric Institute, entitled "Penstocks," gives much material on the design and specification of accepted riveting practice, together with illustrations and tabulations.

6. Welded Steel Pipe. The efficiency of joints in welded steel pipe may be between 90 and 95% provided that the pipe is made in the shop, X-rayed, and stress-relieved. However, for ordinary construction, an efficiency of 80% is recommended for double-welded butt joints. Also, an efficiency of 75% is recommended for double-welded full-fillet lap joints, and the thickness should not exceed ⅜ in. (See the American Welding Society Specifications, D5.1-47, Standard Rules for Field Welding of Steel Storage Tanks.)

All steel pipe welding should be done by the fusion process, which applies external heat by a gas flame or an electric arc to melt the welding electrode and the parent metal. This process differs greatly from the older forged or hammering process of welding. The field of fusion welding of steel pipe, owing to its comparative youth, does not have the long-term, well-established standards of riveting practice. Within a few years, it is presumed that a generally accepted specification for welded penstock construction will be available. The American Welding Society's Rules for Field Welding of Steel Storage Tanks are broadly applicable to pipelines, as are its Standard Qualification Procedure for welding processes and operators.

The current edition of the *Steel Construction Manual* of the American Institute of Steel Construction, New York City, applies to all steel shapes, the recommended working stress being 50% of the yield point given in Table 1.

The method of welding may vary within wide limits. Extensive work has been done both by manual and automatic methods. Two well-known automatic processes are the Unionmelt submerged arc process and the automatic

carbon arc process, both of which are well described in the American Welding Society's *Welding Handbook*. Information on the automatic carbon arc process is found in the literature of the Lincoln Electric Company, Cleveland, Ohio, and information about the Unionmelt submerged arc process in the literature of the Linde Air Products Company, New York, N. Y.

Much more pipe welding is done with the electric arc than with the gas flame. In the electric-arc field, most of the welding is done with direct-current machines because of their greater versatility and because their prime mover may be an electric motor or an internal-combustion engine. Excellent welding can be done both manually and automatically, but automatic welding is often given the preference because of its speed and uniformity. Automatic welding is chiefly a shop practice. In the smaller pipe it is usually possible to move or rotate the work so that flat-position work may be done. However, there is always a considerable amount of work which must be done in vertical and overhead positions.

Though there is no need to review the large field of welding electrodes, it is advisable to consider the use of the coated electrode for better weld quality. The primary reason for the development of coated or shielded arc electrodes was to make possible the deposition of weld metal with physical properties comparable to those of the parent metal The electrode coatings provide a gaseous envelope around the arc and weld metal being deposited, and provide a blanket of molten slag covering the weld. These protect the molten metal from undue oxidation. The extent of the gaseous and slag protection depends on the type of coating. (See the A.S.T.M. Specification A 233-45T "Tentative Specifications for Iron and Steel Arc-Welding Electrodes.")

Wherever possible, longitudinal seams should be shop welded, but there is a limit to the size of the pipe that may be so welded and shipped, on account of railroad clearances. This does not mean that all jobs of large diameter must be pure field work, as an exception to the above would be the Hoover Dam penstock of 30-ft diameter Here, owing to the magnitude of the job, the flat plates (up to 2¾ in. thick) were shipped to the site, where a complete shop with facilities for bending, welding, heat treating, and X-ray analysis was built. This is an unusual example of a combination of field and shop fabrication.

In the design of welded pipe, choice must be made between butt and lap welds. If lap welds are used, a full-fillet weld should be made on each side of the lap. If butt welds are used, they should be of the double-welded type or else of the single-welded butt type with a backing strip. (See American Welding Society's Definitions of Welding Terms and Master Chart of Welding Processes)

The qualifications of welding operators and the periodic testing of them are of paramount importance. (See American Welding Society's Standard Qualification Procedure for Operators) Careful inspection during the progress of the work, together with X-ray examination and cut-out coupon specimens, will help to insure good welding.

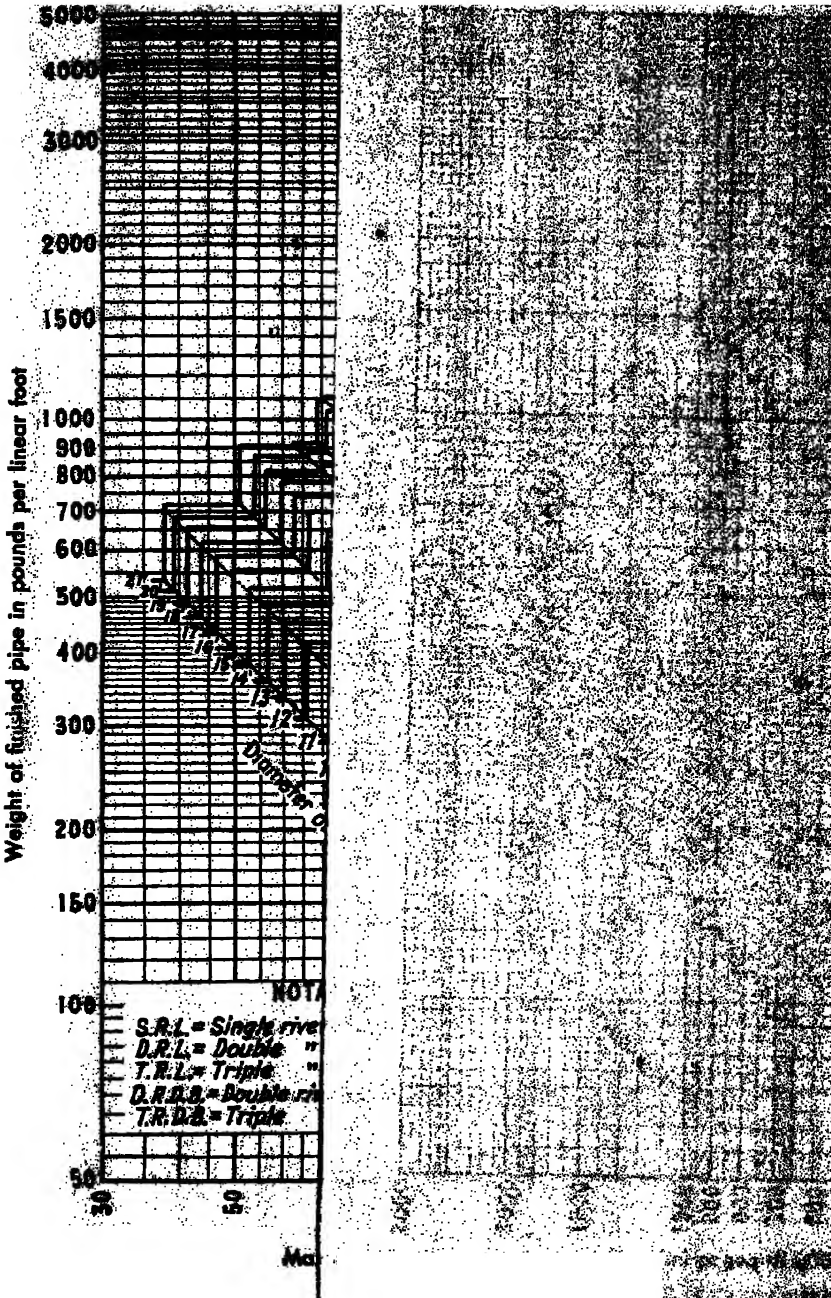
Welded plate work of ASTM-A283-46T material, as listed in Table 1, requires no stress relieving, other than that given mechanically by the first application of hydraulic pressure, unless the thickness exceeds 1¼ in., or, for smaller thicknesses, unless the diameter, in inches, is less than $(120t - 50)$, in which t is the thickness, in inches.

7. Shell Theory of Pipe Design. The shell or membrane theory of pipe design is a relatively new concept. It was first described in an article by

FIG. 2. Ring girders and supports for 18-ft-diameter welded steel pipeline at the Apalachia development, Tennessee. Spacing between supports is 72 ft. (Tennessee Valley Authority.)

K. I. Karlsson of Sweden about 1922 and has been applied to numerous pipe designs in this country since about 1930. This design employs a series of circumferential ring girders which keep the pipe in substantially a true circle and transmit the load to the footings through column supports, as indicated in Fig. 2. In most cases the shell design permits the use of a thinner plate, greater spacing of footings (Fig. 2), and the elimination of stiffening angles longitudinally along the pipe.

The paper entitled "Design of Large Pipe Lines" by Schorer [21] gives an excellent account of the subject, together with complete notation and formulas for computing direct stress on the pipe shell, rim stresses, and stresses in the supporting ring. The section of this article devoted to example and application clearly shows the advisability of increasing the plate thickness at the supporting section for one course width.



Weight of finished pipe in pounds per linear foot
5000
4000
3000
2000
1500
1000
900
800
700
600
500
400
300
200
150
100
50
30
50
Diameter
NOT
S.R.L.= Single rive
D.R.L.= Double
T.R.L.= Triple
D.R.D.B.= Double ri
T.R.D.B.= Triple

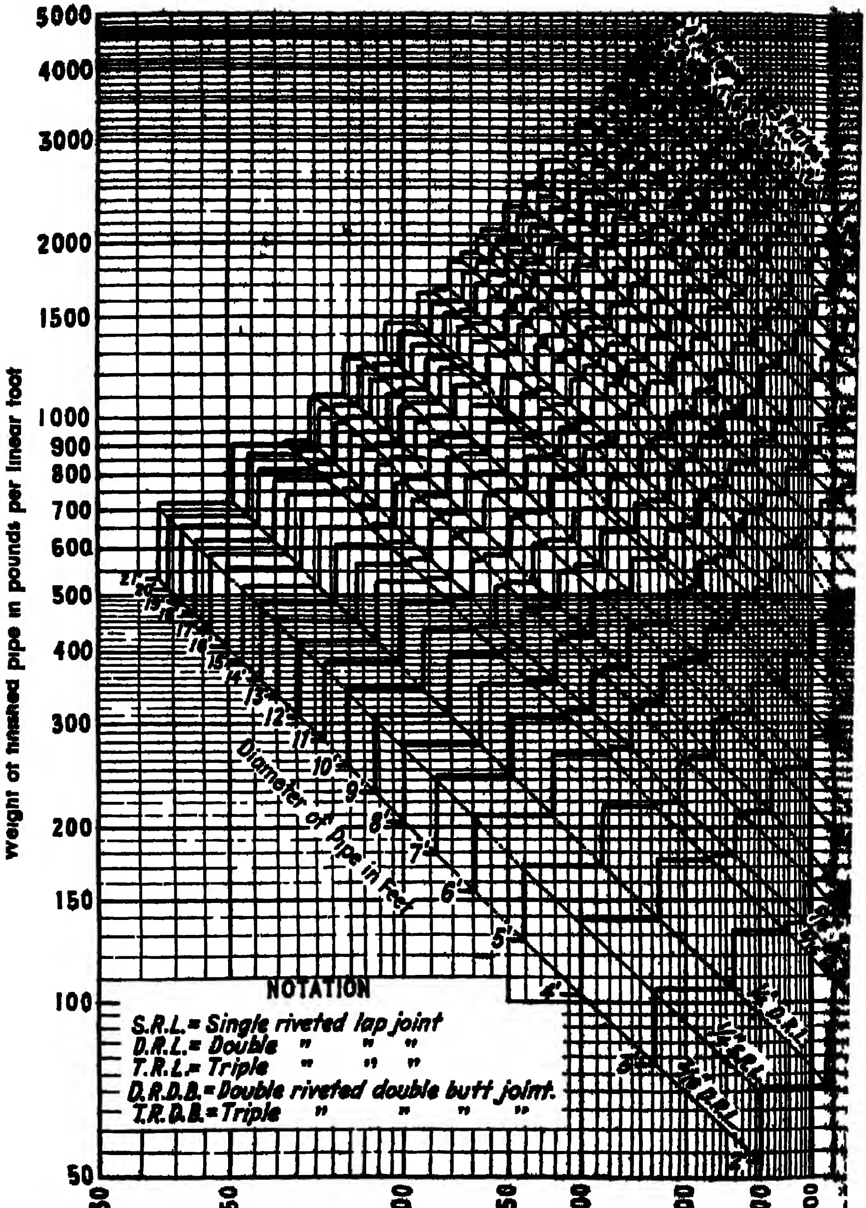


FIG. 3. Diagram for weight of riveted steel pi

(Duplicated for removal.)

8. Estimating Weights of Riveted Steel Pipe without Ring Girders. Figure 3, computed by Gardner C. George, shows the strength, weight, and most economical riveting of steel pipe. From the intersection of the stepped lines, representing diameter of pipe, with the inclined lines representing thickness and riveting, trace vertically to the margin and read the head, or trace horizontally to the margin and read weight of pipe per linear foot. If the thickness and riveting are not desired, trace vertically from the given head to intersect the given diameter, thence horizontally to the margin, and read weight of pipe.

As an example, to show the application of Fig. 3, assume:

Maximum head - 210 ft
Diameter - 10 ft
Allowed stress 15,000 lb per sq in

Locate 210-ft head at the lower margin of the diagram, and trace vertically to intersect the stepped line representing 10-ft diameter; then trace horizontally to read weight of pipe 708 lb per lin ft. The intersection of the 210-ft head line and the 10-ft diameter line lies between the two lines representing $^7\!/_{16}$-in. triple-riveted lap pipe and ½-in. triple-riveted lap pipe. The former would be too weak, as the diagram shows it to be good for only 196-ft head, whereas the latter is good for 215-ft head. Hence the weight given is computed for the stronger pipe.

The diagram may be used for stresses other than 15,000 lb, as follows: In the preceding example, suppose a stress of 20,000 lb is allowed. Then enter the diagram with a head computed as follows, and proceed as already described.

$$\text{Head to use in diagram} = 210 \times \frac{15{,}000}{20{,}000} = 158 \text{ ft head}$$

9. Estimating Weights of Welded Steel Pipe without Ring Girders. The weight of welded steel pipe with butt joints and no backing strips or lap can be obtained from the following formula:

$$W = \frac{8.17whd^2}{ES} = \frac{1000hd^2}{3ES} \qquad [2]$$

where W = weight of pipe, in pounds per linear foot;
w = 40.8 lb; = weight of a 12 × 12 × 1 in. piece of pipe material;
h = pressure at the center line of the conduit, due to all estimated loadings, in feet of water;
d = internal diameter of pipe, in feet;
E = joint efficiency (80% recommended for double-welded butt joints in all thicknesses, and 75% for double full-fillet lap joints in thicknesses not to exceed ⅜ in.; efficiencies are for field-welded pipe);
S = unit working stress, in pounds per square inch.

The design of ring girders varies with each project, hence no empirical formula is given for estimating their weight

10. Piers. A steel pipeline may be designed either with saddles, as indicated in Fig 4, or with ring girders supported by short columns, as indicated in Figs 2 and 5, but the weight of the pipe and its contents is transmitted to the ground through concrete piers

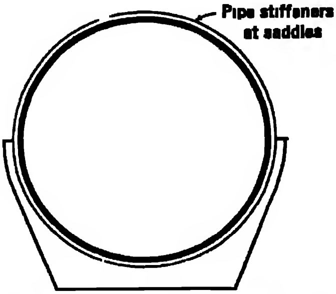


FIG 4 Schematic saddle support

Concrete piers in earth are usually placed about 3 or more feet into the ground They should always be designed to correspond to the bearing power of the soil and the weight of the pipe and its contents It is not necessary to place piers below the frostline, as heaving due to frost action will not damage the pipe

For rock surfaces, the concrete piers are merely a coating of concrete to smooth up the rock Figure 6 indicates a type of concrete pier for steep exposed pipes for rock foundations

11. Spacing of Supports. In general, piers, discussed in Section 10, whether used with saddles or ring girders, may be placed as far apart as beam

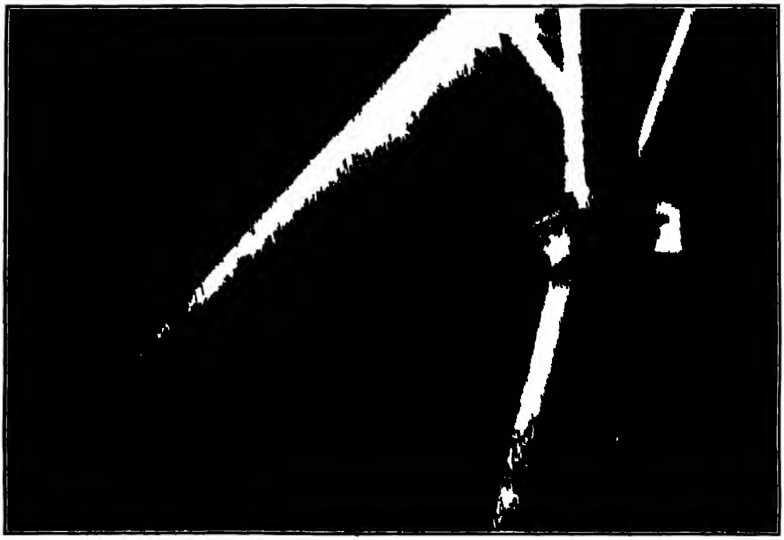


FIG 5 Ring girders and bents for 9 ft-0 in diameter welded steel pipeline at the Nantahala development North Carolina Maximum span is 80 ft

action in the pipe and supporting power of the soil or other medium will allow For pipelines supported on saddles there are some combinations of saddle arc, shell thickness, shell diameter, and pier spacing requiring no shell stiffeners,

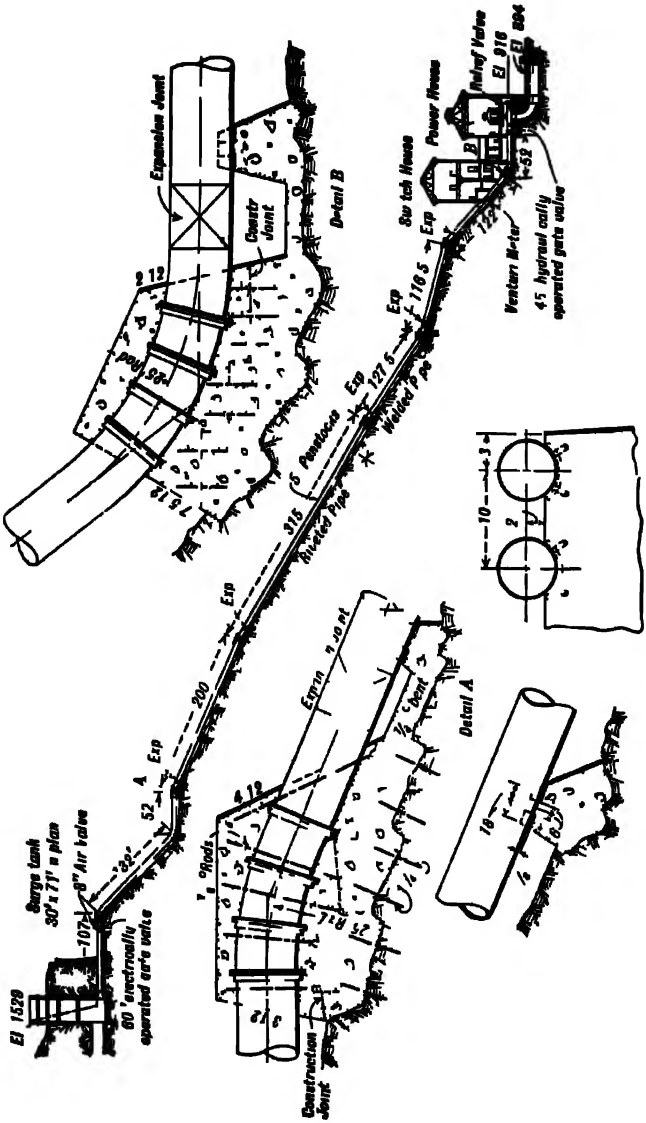


FIG 6 Anchors and piers for the Tallulah Falls Development Georgia Railway and Power Co (*Eng News-Rec*, Vol 69 p 361)

but there are other combinations requiring circumferential stiffeners near the saddles only. For thin shells ring-girder design is usually preferable to saddle design, provided the foundation conditions are adequate, as it permits longer spans with no stiffeners other than the ring-girder supports, and it requires fewer piers.

Typical examples of ring-girder design are: the 9-ft-diameter welded steel pipeline at the Nantahala development, North Carolina, having a maximum span of 80 ft, as indicated in Fig. 5, and the 18-ft-diameter welded steel pipeline at the Apalachia development, Tennessee, as indicated in Fig. 2, where the span is 72 ft.

There is no natural limit for a safe, unsupported span. The portion of longitudinal stress due to liquid weight is independent of the radius of the pipe but varies directly as the square of the span and inversely as the thickness. The portion of the longitudinal stress due to metal weight is independent of the thickness but varies directly as the square of the span and inversely as the radius.

12. Anchorages. (*a*) *Location.* It is customary to install anchorages at angle joints in the pipeline and between every two expansion joints in a long pipeline. Anchorages are not usually provided for buried pipe unless the pipe is to be previously filled for testing. Anchors should be designed so that they will, by gravity alone, take care of the resultant of all forces. Rods for anchorages should be used only in the hardest and most solid ledge rock.

Figure 6 gives details of a typical anchor. Where anchors are located on tangents and at the intake, several circumferential angles, riveted or welded to the pipe, serve to transfer the forces from the pipe to the anchors. Anchors at sharp bends seldom need this provision, as the anchorage should envelop all or part of the bend.

(*b*) *Analysis of Stresses for Anchors.** The following formulas and diagrams give a complete analysis of all the stresses acting upon an anchor. Many of these forces will generally be of small magnitude, but they have been included in this discussion in order to make it complete. As shown on the diagram, Fig. 7, all these forces acting at the bend are finally combined into vertical and horizontal components, which, in turn, combined with the weight of the anchor itself, give a resultant that must be within the middle third of the anchor base.

Definition of Symbols:

h = static head at any point;
A = inside area of pipe, in square inches;
p = pressure, in pounds per square inch, at any point;
a = angle of upstream leg with horizontal;
b = angle of downstream leg with horizontal;

* From *Penstocks,* a Report of Hydraulic Power Committee, Edison Electric Institute, 1936.

c = exterior horizontal angle between center lines of upstream and downstream legs;
d = true deflection angle of bend;
W = weight of pipe from a point midway between anchor and first pier upstream to expansion joint;
W' = weight of pipe from a point midway between anchor and first pier downstream to expansion joint;
P = weight of water in pipe W;
P' = weight of water in pipe W';
Q = weight of water and pipe between points midway between adjacent piers above and below anchor;
f = friction coefficient of pipe on piers (this should be 0.6 for untreated contacts as steel on steel or concrete, 0.4 to 0.5 for lubricated surfaces, and much less for roller or hinged bearings);
V = velocity of water, in feet per second;
C = circumference of pipe, in feet, at expansion joint above anchor;
C' = circumference of pipe at expansion joint below anchor;
S = weight of pipe from center line of anchor to point midway between anchor and adjacent upstream pier;
S' = weight of pipe from center line of anchor to point midway between anchor and adjacent downstream pier;
F = friction in expansion joint per linear foot;
E = area of exposed end of pipe at expansion joint, in square inches;
R = area of pipe in square feet;
w = weight of water, per cubic foot;
g = acceleration of gravity;
K = reduction in area at reducer above anchor;
K' = reduction in area at reducer below anchor.

Penstock Anchor Forces:

1. Friction on supports due to expansion or contraction on upstream side of anchor = $f(W + P) \cos a$.
2. Friction on supports due to expansion or contraction on downstream side of anchor = $f(W' + P') \cos b$.
3. Friction of expansion joint on upstream side of anchor due to expansion or contraction = CF (F = 500 lb per lin ft, determined by experiment).
4. Friction of expansion joint on downstream side of anchor due to expansion or contraction = $C'F$.
5. Force due to dead weight of pipe acting parallel to center line of pipe on upstream side of anchor = $(W + S) \sin a$.
6. Force due to dead weight of pipe acting parallel to center line of pipe on downstream side of anchor = $(W' + S') \sin b$.
7. Force due to bend in pipe with water flowing = $RVwV/g$ (divided by the radius of the bend).

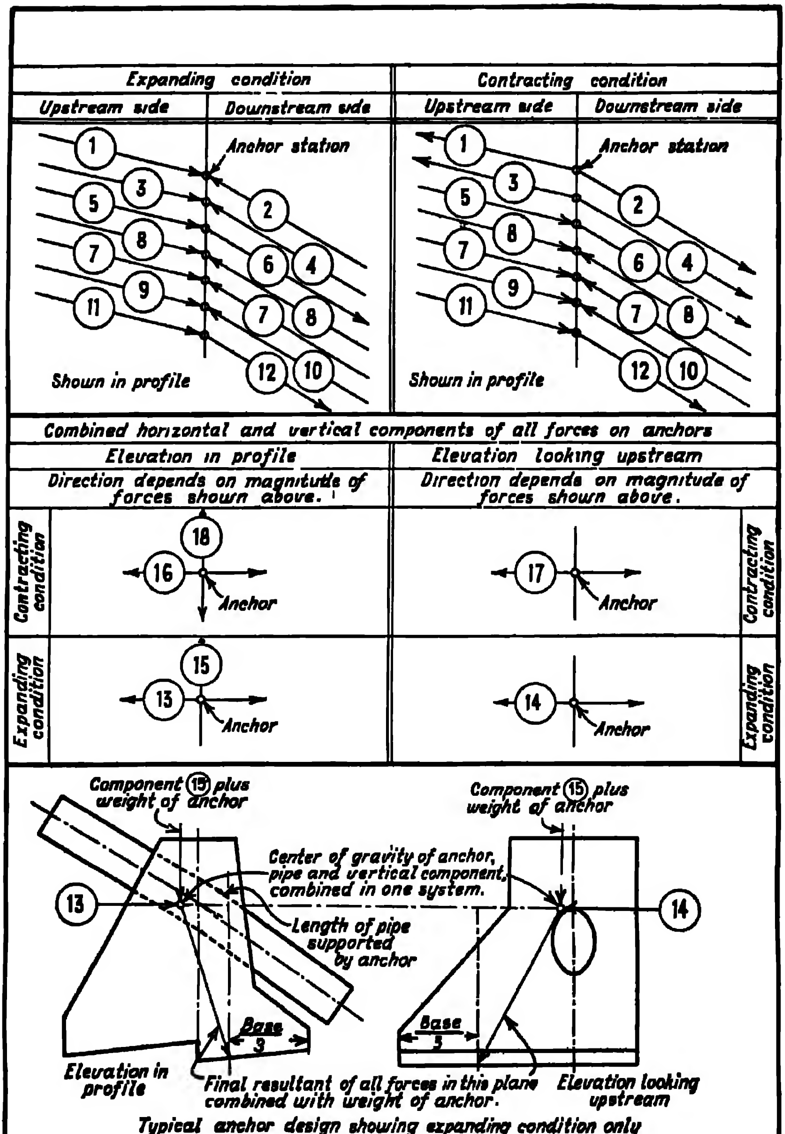


FIG. 7. Diagrams showing exterior forces on anchors.

8. Hydrostatic force at anchor = pA (acts along axis of pipe toward bend on each side of anchor.
9. Force on exposed end of expansion joint, upstream side of anchor = Ep.
10. Force on exposed end of expansion joint, downstream side of anchor = $E'p$.
11. Force due to reducer above anchor = pK.
12. Force due to reducer below anchor = pK'.
13. Horizontal component in plane of upper penstock center line for algebraic sum of all forces on anchor, expanding condition.
14. Horizontal component normal to plane of upper penstock center line for algebraic sum of all forces on anchor, expanding condition.
15. Vertical component for algebraic sum of all forces on anchor, expanding condition.
16. Horizontal component in plane of upper penstock center line for algebraic sum of all forces acting on anchor, contracting condition.
17. Horizontal component normal to plane of upper penstock center line for algebraic sum of all forces on anchor, contracting condition.
18. Vertical component for algebraic sum of all forces on anchor, contracting condition.

13. Expansion Joints. Some means are generally necessary for taking care of expansion and contraction due to changes in temperature in exposed steel pipes. If the pipeline is flat, the expansion joints are usually located midway between the anchorages, in order to reduce the movement of the pipe over the piers to a minimum. On steep slopes, however, the anchorage at the crest of the hill is frequently more difficult to install than the one at the foot of the hill. For this reason, it is advantageous, in such cases, to locate the expansion joint near the uphill anchor, thus transferring all the friction on the piers to the lower anchor. This arrangement also facilitates the erection of the pipe, as it is usually difficult on steel pipe to erect that portion between the expansion joint and the upper anchorage, on account of the tendency of the pipe to slide down the hill. Additional expansion joints are also required if the distance between anchorages is so great that the movement of the pipe, due to extremes of temperature, is greater than a single expansion joint can accommodate.

Short pipes with frequent bends are sometimes considered flexible enough for this purpose, but some form of expansion joint is usually necessary.

The use of expansion joints for buried pipe is seldom necessary, as such pipe is not exposed to great extremes of temperature. However, if a buried pipe is solidly retained at each end, has no bends to accommodate movement, and has inadequate circumferential rivets, the omission of an expansion joint may cause failure.

Expansion joints eliminate excessive longitudinal stress in the pipe and thus permit of the use of lighter circumferential joints.

There are two general types of expansion joints, i.e., the slip joint and the diaphragm joint. Figure 8 is a slip joint of the type frequently used on welded pipelines. Figure 9 is a typical slip joint built up of riveted steel pipe. This particular joint has one surface copper-plated, to obviate the possibility of its rusting tight.

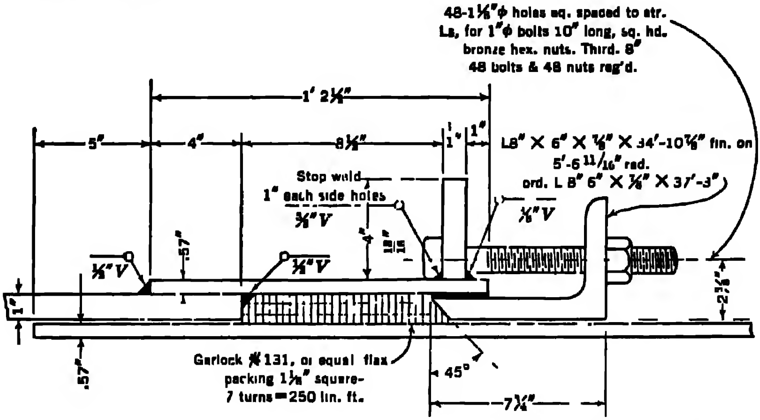


FIG. 8 Slip joint type of expansion joint for welded steel pipelines.

The packing of the stuffing box of slip joints can be satisfactorily done with square-braided hemp or flax packing impregnated with graphite. Joints of this type, when properly packed, give entire satisfaction and have insignificant leakage. They require some attention, however, and are thought by some to be inferior, under certain conditions, to the diaphragm joint.

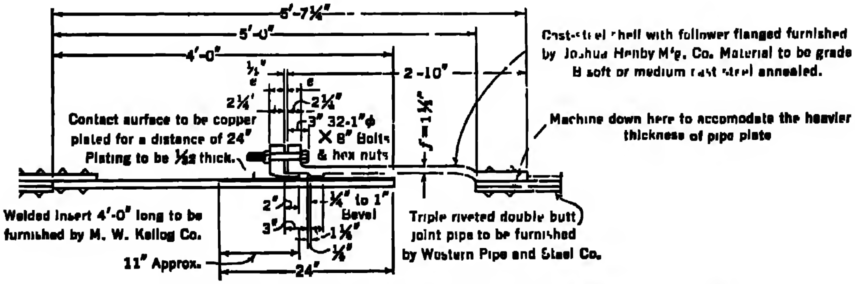


FIG. 9. Slip joint type of expansion for riveted steel pipelines.

The conditions favorable to diaphragm joints are slight movement and low head. A typical diaphragm or bellows joint is shown in Fig. 10. The movement of the pipe is provided for by the flexibility of the steel in the joint. In very cold climates the expansion joint should be boxed in to prevent freezing (Section 15). Such joints are not particularly well adapted for heads

greater than about 200 ft because, if they possess the stiffness required to withstand internal pressure under greater heads, they are not flexible enough for expansion joints.

The expansion and contraction of steel penstocks due to temperature changes is greatest when the penstock is empty. As the penstock is likely to remain empty through a wide range of temperature changes during the construction period, expansion joints should be designed to take care of these conditions. On the Catskill Aqueduct a number of determinations were made of the actual expansion in steel pipelines, due to temperature changes. Table 3 shows that the actual expansion agrees closely with the computed theoretical expansion. The theoretical expansion was computed with a coefficient of 0.0000072, and the temperature used was the mean atmospheric temperature, inside and outside the pipe.

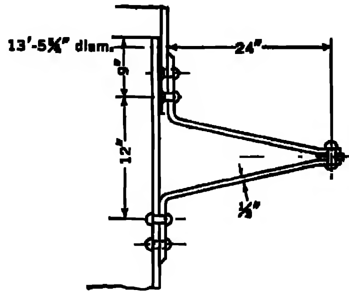


Fig. 10. Bellows type of expansion joint for steel pipe.

TABLE 3

Temperature Changes in Steel Pipe—Pipes Empty and Exposed *

Diameter, feet and inches	Length, feet	Temperature Change, degrees Fahrenheit †	Expansion, inches	
			Observed	Theoretical ‡
8-10	3078	15	3	$3\frac{1}{2}$
8-10	3078	20	4-5	$4\frac{3}{4}$
8-10	3078	25	6-7	6
8-10	3078	30	7	$7\frac{1}{4}$
8-6	807	19	$1\frac{1}{4}$	$1\frac{1}{4}$
8-6	807	30	$2\frac{1}{8}$	2
8-6	1292	15	$1\frac{3}{4}$	$1\frac{5}{8}$
8-6	1292	33	$3\frac{5}{8}$	$3\frac{3}{8}$

* From Mansfield Merriman, "Building Additional Siphons of Catskill Aqueduct," *Eng. News-Rec.*, Vol. 90, p. 869.

† Mean of atmospheric temperature, inside and outside of pipe.

‡ Theoretical expansion computed with coefficient of 0.0000072.

14. Painting Steel Pipes. Numerous paints for steel pipes are on the market, but none of them give a permanent protection to the steel. The life of a steel pipe aboveground can be prolonged indefinitely by frequent painting. The outside surface of a buried pipe is inaccessible except at a great expense. For either pipe it would be economical to pay several times the present market price for the best paint if a permanent protective coating were available.

The relative merits of the different paints on the market cannot be discussed here, but under no circumstances should the quality of paint for steel pipe be sacrificed for a saving in cost.

The most rigid inspection of painting should be made. The best paint poorly applied is no better than a paint of very inferior quality, and frequently not so good.

The paint to be adopted depends upon the location of the surface to be covered, i.e.,

(*a*) the outside of the pipe aboveground;
(*b*) the outside of the pipe underground;
(*c*) the outside of the pipe in dark, damp compartments;
(*d*) the inside of the pipe.

Each of these divisions is subject to different paint-disintegrating influences and usually requires a different class of coating. For important pipelines, the paint should be purchased only from those manufacturers who have had long experience, unless an exhaustive test of the paint is to be made.

For buried pipe, care should be taken that the backfill does not contain tannic acid from decayed vegetable growth, or other ingredients that may affect the durability of the paint. The soil should be subjected to rigid chemical analysis by the engineer and by the chemists of the paint manufacturer.

Before starting the first coat of red lead or whatever is used, the surface of the pipe to be painted should be sandblasted clean of rust and scale. This is the most important prerequisite for successful painting of steel. Nothing makes a better first coat than high-quality red lead, but it does not stand abrasion well. Consequently, the second and third coats should be some good-quality underwater paint. Aluminum paint has been successful for both inside and outside the pipe. For outside use, it has the additional advantage of reflecting the sun's rays.

Bitumastic compound has been successfully used. The additional expense is almost always worth while for the inside of the pipe. For the outside, however, the use of this expensive material is questionable as the outside may usually be repainted much more readily. Bitumastic compound, as made and applied by the Wailes Dove-Hermiston Company, has been used for 30 years on the gates of the Panama Canal with complete success. Its only disadvantage is its cost, which is often twice as much as that of ordinary paint.

Another paint for the inside of pipe is Sipes' Subway Black. In one power plant in Virginia, Sipes' Subway Black was used in the pipeline and after 9 years the coating was in almost as good a condition as when first applied.

The engineer should bear in mind that usually the only chance he will ever get, during the life of the hydro plant, to do a good job of painting the inside of the pipe is before the plant is placed in service.

Plastic coatings, applied either hot (preferably) or cold, may have considerable merit and have been used with success on ship hulls. Where leak-

age current or other causes would produce electrolysis, the protection of the pipe by cathodic means is necessary.

Where conditions are conducive to excessive internal corrosion, the expense of a nickel-clad penstock may be justified. The lower hydraulic friction loss and the considerable savings in maintenance will usually offset the higher first cost. (See Ref. 2, Section 17.)

15. Protection against Freezing. In very cold climates, an ice sheet will form on the inside of exposed steel pipes unless the pipe is short or the velocity high. The ice sheet may become sufficiently thick to affect the hydraulic properties of the pipe, and during a thaw enough ice may break away and enter the turbine to plug the gates. Water hammer in such cases may be considerably greater than that computed for normal operation. Whether or not trouble will be experienced from ice depends upon the following factors:

(*a*) the velocity during low-load periods;
(*b*) the duration of such periods;
(*c*) the climate at the site;
(*d*) the exposure of the pipe to prevailing winds;
(*e*) the depth of the forebay;
(*f*) the depth of the pipe entrance below water surface.

Therefore, the probability of ice troubles must be estimated according to experience and comparison with existing plants in the vicinity. Pipes over 100 ft long, in a climate similar to that of the mountainous regions of northern New York, operating at usual velocities and at about 50% load factor on a general power and lighting load, are likely to give occasional trouble unless protected.

Pipes may be protected from freezing by burying them or covering them with insulation, or wood concrete housing.

There is practically no danger from freezing in a pipe covered with 3 ft of earth, even though the frostline is twice this distance below the surface, provided the center line of the pipe is below the frostline.

16. Buried Pipe versus Exposed Pipe. The authors recommend exposed pipe unless the local conditions make the use of buried pipe advisable. The advantages of exposed pipe are as follows:

(*a*) It provides more room for construction.

(*b*) Such pipe is more accessible for inspection, maintenance, and repairs This is the predominating feature favorable to exposed pipe.

(*c*) Where pipe is buried, inspection and renewal of paint on the outside of the pipe are not likely to occur frequently, and buried pipe may, therefore, be considered to have a shorter life because of corrosion.

(*d*) Exposed pipe is usually less expensive to install.

The conditions favorable to buried pipe are as follows:

(*a*) Pipes extending down steep hillsides on earth foundation make anchoring and supporting very difficult. If the pipe is buried, anchors and supports are not usually necessary.

(*b*) On steep side-hill locations there is frequently danger from landslides, snowslides, and falling rocks, which would injure the pipe unless buried.

(*c*) In cold climates, where the pipe is very long and the velocity low, it is sometimes less expensive to bury the pipe than to provide other means of protection against freezing.

(*d*) Where the pipe passes through earth cut, it is often cheaper to bury it with the excavated earth than to provide cradles and sills.

Pipe that is to be buried is generally supported in the trench, temporarily, on blocking. In refilling the trench, the material should be carefully tamped into place as otherwise the material under the bottom of the pipe is very likely to settle, leaving a space of several inches under the pipe.

Experience has shown that the use of porous material around buried pipe, allowing free drainage, results in longer life of the pipe than the use of impervious clays. The characteristics of the soil surrounding the pipe should be carefully studied, as certain soils are very detrimental to most pipe coatings (see Section 14).

Pipe that is only partly buried usually corrodes quickly at or just below the ground line, owing to the extreme variation in temperature and moisture.

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CHAPTER 32

WOOD-STAVE PIPE AND CONCRETE PIPE

1. General. A general outline of the purpose and use of wood-stave pipe and concrete pipe in hydroelectric practice and a discussion of those features that are common to all conduits are presented in Chapter 28.

A. Wood-stave Pipe

By Byron E. White *

2. Types of Wood-stave Pipe. Wood-stave pipe consists of a shell formed of wooden staves bound together by steel bands. The bands are designed to withstand the bursting pressure of the water within the pipe. There are two general types of wood-stave pipe, designated, according to the method of manufacture or assembly, as (1) machine-banded wood-stave pipe, and (2) continuous wood-stave pipe.

Machine-banded Wood-stave Pipe. Machine-banded wood-stave pipe is a manufactured product and is transported to the job as a finished pipe in lengths from 6 to 20 ft, as desired. It is regularly made in sizes from 2 to 24 in. in diameter and for heads up to 400 ft. It has been made for heads up to 750 ft and up to 42 in. in diameter for use under low pressures.

Machine-banded wood-stave pipe is suitable for the conveyance of only relatively small quantities of water, on account of its restricted size. It can be made to withstand heads up to 500 ft or more in the small sizes. This type of pipe has been used very extensively for water mains for domestic water supply in the West, and also for conveying water for irrigation, hydraulic sluicing, etc., as well as for sewers and drains.

Continuous Wood-stave Pipe. Continuous wood-stave pipe is built up in place where it is to be used. The lumber is delivered in the form of staves, together with the steel bands and other accessories.

One of the particular advantages of continuous wood-stave pipe is the relatively small size and weight of the individual parts, which permits the transportation of single staves and bands to the site where the pipe is to be erected. This is a special advantage in rough, mountainous, or wooded country where transportation is difficult and material may have to be carried to the site by pack animals or men.

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Continuous wood-stave pipe has been constructed in diameters from 6 in. up to 16 ft. Some of the pipe manufacturers are prepared to quote on pipes up to 20 ft or more in diameter. The maximum heads possible for this type are shown in Fig. 4 for pipes from 3 ft to 22 ft in diameter.

Continuous wood-stave pipe, on account of its flexibility, ease and rapidity of construction, comparative ease of transportation, and its relatively low cost at lower heads, as contrasted with other types of pipe, has a distinct field of usefulness in connection with hydroelectric power stations. Within certain limitations of head, wood-stave pipe is cheap compared to other pipes if properly installed, and particularly if the staves are treated to prevent decay. Thus it is especially suitable for power developments where it is essential to keep capital expenditures down to the minimum.

Relative Costs. The fields of continuous and machine-banded pipe overlap somewhat, and the choice between them is usually made on the basis of ease and cost of transportation. For instance, on account of its greater bulk, transportation costs for the larger sizes of machine-banded pipe are much greater than for continuous pipe where long freight hauls are involved. For the same reason, there are also many cases where continuous pipe material may be transported to the site of the work much more easily than machine-banded pipe sections in the larger sizes.

3. Machine-banded Wood-stave Pipe. A typical machine-banded wood-stave pipe is shown in Fig. 1. The staves are assembled on a form,

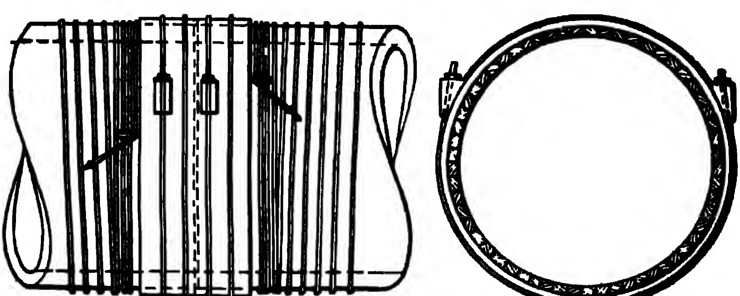


Fig. 1. Typical banded collar for wire-wound pipe

in a machine in which they are wound spirally with galvanized round steel wire, or a galvanized flat steel band, which is fastened to the wood by a special clip at one end of the pipe section, wound on under the proper tension, and again fastened with a similar clip at the other end of the section of pipe, and cut off. The two ends of the section are then turned down square, if separate collars are to be used; or a recess is formed at one end and a tenon on the other end. After this the section is first rolled through a bath of asphaltum and then through a bed of sawdust, which absorbs the excess asphaltum, leaving the pipe in such condition that it can be handled after drying.

Machine-banded wood-stave pipe is made up in random lengths, which may be from 3 or 4 ft to 20 ft long, in order to make the best use of the available lumber of suitable quality.

The end joints between adjacent lengths are made in two ways: (1) by means of separate collars encircling the ends of adjacent sections of pipes (Fig. 1), and (2) by means of a recess and tenon or slip joint (Fig. 2).

The use of the slip-joint or "inserted-joint" type (Fig. 2) of pipe is restricted to low-head work and to cases where a slight amount of leakage is not objectionable, as, for instance, irrigation work, hydraulic sluicing, etc. The maximum head to which this type of pipe is suited is about 100 ft.

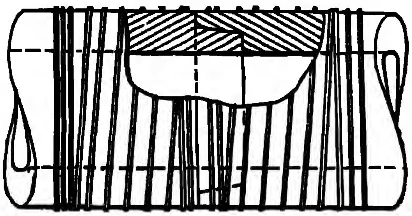


FIG. 2. Recess-and-tenon type or inserted joint type of wire-wound pipe joint.

If the design has square ends and collars, the hoops or wire banding is left off for a suitable distance from each end, and adjacent sections are connected by means of wood-stave, steel, or cast-iron collars, which are made to fit snugly around the ends of the adjacent sections. The wood-stave collars are made 6 to 8 in. in length, and in the smaller sizes, up to about 12 in. in diameter, are machine-banded in the same manner as the pipe. Collars 6 in. in length are used for diameters up to 12 in. For the larger sizes, one-piece hoops or bands, provided with shoes, nuts, and washers, are used for tightening up the collars. Not less than three bands should be used per coupling, and these require "cinching" or tightening after the pipe has been erected.

Wood-stave collars should always be creosoted as they are not subject to complete saturation from the water within the pipe. Many cases can be cited of untreated collars' having failed from decay.

In the recess-and-tenon type of pipe, the tenons are made of such dimensions as to fit tightly into the recessed ends, and each section is assembled with the recessed end outward, and driven home by means of a wooden follower and ram.

This type of wood-stave pipe, unless of large diameter and under low heads, or laid on unstable ground, does not usually require special supports. It may be assembled directly on the ground, and may be laid to curves of rather long radius, depending on the shortness of the sections and the diameter.

Tables 1 and 2 give the approximate ranges of size, weight, and wire banding, for this type of pipe, as made by a prominent manufacturer.

4. Continuous Wood-stave Pipe. Continuous wood-stave pipe is assembled, stave by stave, by either of two methods: (1) It may be assembled in the permanent cradles in which it is to rest. (2) In the absence of fully semicircular permanent cradles, temporary portable wooden cradles

TABLE 1

RANGE OF SIZES AND THICKNESS OF STAVES FOR MACHINE-BANDED WOOD-STAVE PIPE

Inside Diameter, inches	Thickness of Staves, inches	For Heads Less than Feet	Thickness of Staves, inches	For Heads Over, Feet
2	1			
3	1			
4	$1\frac{1}{16}$			
5	$1\frac{1}{16}$			
6	$1\frac{1}{8}$			
8	$1\frac{1}{8}$			
10	$1\frac{1}{8}$	250	$1\frac{1}{4}$	250
12	$1\frac{3}{16}$	250	$1\frac{1}{4}$	250
14	$1\frac{3}{16}$	250	$1\frac{1}{4}$	250
16	$1\frac{1}{4}$	200	$1\frac{5}{16}$	200
18	$1\frac{1}{4}$	200	$1\frac{5}{16}$	200
20	$1\frac{5}{16}$	150	$1\frac{3}{8}$	150
22	$1\frac{5}{16}$	150	$1\frac{3}{8}$	150
24	$1\frac{5}{16}$	150	$1\frac{3}{8}$	150
26	$1\frac{3}{8}$	150	$1\frac{1}{2}$	150
28	$1\frac{3}{8}$	150	$1\frac{1}{2}$	150
30	$1\frac{3}{8}$	150	$1\frac{1}{2}$	150
32	$1\frac{3}{8}$	150	$1\frac{1}{2}$	150

NOTES. Maximum spacing of wire, 8 in. Working stress of wire, 15,000 lb per sq in. Maximum pressure head, 400 ft approx. Range of lengths of pipe sections, 6 ft to 25 ft.

may be used for setting up the lower half of the pipe. After this, the top half is laid up on two collapsible, portable wooden forms in the interior of the pipe, which, with the portable cradles, are moved forward for setting up successive lengths.

In the usual method of construction, staves of approximately uniform length are used for each section, with alternate staves projecting 2 or more feet beyond the adjacent ones. The staves of the succeeding section are driven into the spaces between these projecting staves, thus tying the pipe together and providing continuous beam action between supports. This method of assembly is shown in Fig. 3. Continuous pipe is sometimes assembled with staves of random lengths, no attempt being made to keep the ends of alternate staves approximately together, longitudinally. The only

TABLE 2

Weights of Untreated Machine-banded Coupling Type Wood-stave Pipe, and Sizes and Spacing of Wire

Size, inches	50-foot Head			400-foot Head		
	Weight, pounds	Foot-size, number	Wire-spacing, inches	Weight, pounds	Foot-size, number	Wire-spacing, inch
2	3.3	8	3	4.1	8	$\frac{3}{4}$
4	5.7	8	3	7.3	8	$\frac{11}{16}$
6	8.5	6	3	11.6	6	$\frac{11}{16}$
8	10.8	6	3	15.1	6	$\frac{5}{8}$
10	13.7	4	3	21.3	4	$\frac{11}{16}$
12	16.8	4	3	27.0	4	$\frac{9}{16}$
14	19.6	4	3	34.3	2	$\frac{5}{8}$
16	24.1	2	3	41.7	2	$\frac{9}{16}$
18	26.8	2	3	48.7	2	$\frac{1}{2}$
20	30.6	2	3	58.4	2	$\frac{7}{16}$
22	33.6	2	3	64.3	1	$\frac{1}{2}$
24	36.4	2	3	74.3	1	$\frac{7}{16}$

necessary precaution to observe is that there should be at least a 2-ft break in all longitudinal joints; the break, of course, may be of any greater length.

The staves are bound together by circular bands of steel of such cross-sectional area and so spaced along the pipe as to resist most efficiently the bursting pressure within.

In setting up the pipe, all bands finally required over a support are put in place, together with sufficient intermediate bands, approximately 2 ft apart on centers, to hold the pipe together. When a suitable length of pipe has been set up, the remaining bands are placed and all are cinched up, preparatory to filling the pipe with water.

This type of pipe derives its strength against bursting from the steel bands which encircle the pipe and by which it is made tight. The ability of the pipe to stand up without collapsing when empty depends on the thickness of the staves and the strength of the bands. It has been found in practice that certain thicknesses of staves are most suitable for certain pipe diameters, heads, and conditions. These matters will be further discussed in succeeding sections.

Owing to the flexibility of the staves, it is possible to bend continuous wood pipe to fit vertical, horizontal, and inclined curves within certain

limits which are specified in Section 10. Hence, it is possible within these limits to construct such pipe continuously without the introduction of special bends.

The pipe is made tight by "cinching," or tightening down the nuts on the bands. In erection, it is an advantage to have the staves as dry as possible so that little or no further drying out from the effects of sun and wind will take place after the pipe has been erected. If the staves are not dry enough, constant watchfulness is required and the bands must be tightened up whenever the pipe shows signs of weakness from drying out. After the

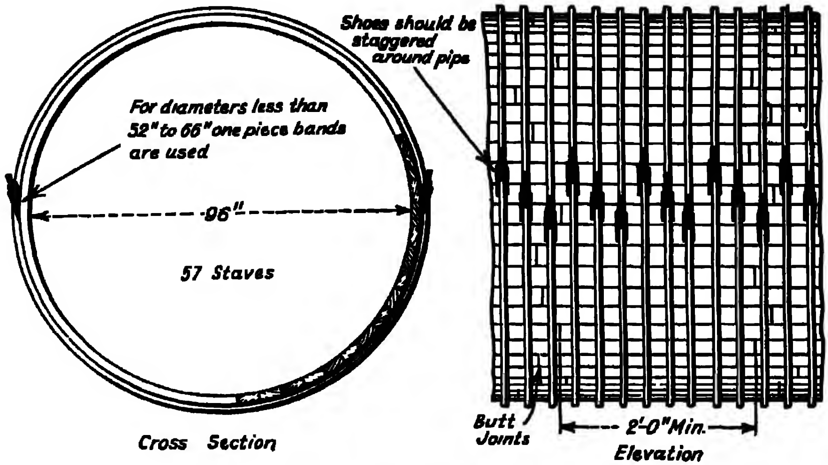


Fig. 3. Typical continuous wood-stave pipe.

pipe has been completed and tied in to other structures, filling with water should take place slowly, as, in spite of all precautions, it is impossible to cinch up the bands tightly enough to prevent some leaks, and it is desirable that such leaks should take place under as little hydrostatic head as possible, so as to limit the amount of water leaking out of the pipe. Excessive leakage might cause erosion and endanger the safety of the pipe.

A further reason for slowly filling with water is to reduce the total stress in the bands during the time that the staves are becoming saturated, with the consequent swelling of the wood and temporary increase in band tension, as described in Section 9.

As soon as the water comes in contact with the wood, some water is absorbed, causing the staves to swell slightly. This action usually closes practically all leaks within a few hours. Those leaks which do not close quickly or do not close after the pipe has been re-cinched have to be dealt with by means of wedges or plugs.

Where a small leak exists between adjacent staves, a carpenter's chisel is driven in about ½ in. from the edge of one of the staves, and a very thin, wide wedge of softwood, usually of white pine if obtainable, is driven into the

opening thus made, forcing the edges of the staves tightly together. This is generally sufficient to close all but the most serious leaks. The wedge should be cut off flush with the outer surface of the pipe. Sometimes it is necessary to drive softwood plugs into small holes where wedging in the above manner is not sufficient. Wedges should never be driven into the longitudinal seams, as they tend to spread the staves apart and extend the leak.

The sections that follow pertain principally to continuous wood-stave pipe and to the special requirements in its design, installation, and maintenance. Much of the matter is equally applicable to machine-banded pipe.

From the information at hand, it appears that wood-stave pipe has in the past been largely designed and installed by the exercise of judgment and the use of rule-of-thumb methods rather than by the application of theory. On some important points, little engineering information has been published. From the experience of many years with successful and unsuccessful installations, certain principles and practices which approximate standards have been evolved. The observance of these will result in a safe and conservatively designed installation.

The data in regard to design have been collected from the best available sources and represent conservative present practice. Several design curves were developed from such information in order to present it in more usable and rational form.

5. Staves. *Cross-section of Staves.* Staves are formed in the mill so that the interior and exterior surfaces are arcs of circles, the interior having a radius equal to one half of the nominal pipe diameter. The object in making the exterior surface of the staves truly circular is to distribute the bearing stresses between the bands and the staves equally around the circumference and to take full advantage of the band strength and the bearing value of the wood. A polygonal exterior surface, which would result if plane-outer-surface staves were used, would concentrate the bearing stresses at and near the edges of staves, reducing the maximum head that might be impressed on the pipe and also inducing checks in the wood. The edges of the staves are customarily made radial so that adjacent staves may make full contact with each other throughout their entire surfaces.

Some engineers have specified that one side of each stave should be milled to a plain radial surface, with a bead $\frac{1}{16}$ in. or $\frac{1}{8}$ in. high on the other radial side. There is considerable doubt as to the advantage of the beaded stave, as it is thought by some that the crushing of the bead may possibly hasten decay.

The plain radial joint, when properly installed and cinched up, has been found to make a satisfactorily tight pipe.

Thickness and Width of Staves. The dimensions of staves depend principally upon the following requirements:

1. Staves should have sufficient structural strength to resist the tendency to go "out of round" when filled with water, and to bridge properly between cradles without sagging.

2 Staves should be thin enough, within the limits imposed upon them to be thoroughly saturated when under working head or pressure if they are not treated with preservative

3 The width of stave is dictated partly by considerations of economy and the use of stock material and partly by the diameter of the pipe

A compromise has to be effected between requirements 1 and 2, as they lead in opposite directions

The thicknesses of staves shown in Fig 4 have been found from experience to be suitable for the designated sizes of pipe and heads under varying conditions There is at least one formula in existence which indicates much

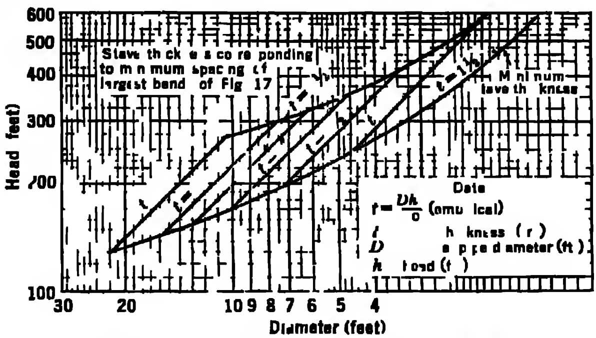


FIG 4 Diagram showing the thickness of staves for wood-stave pipe used in present practice

smaller thicknesses but experience indicates that a strong satisfactory pipe requires the use of the thicknesses herein referred to

Figure 5 gives the weight of staves for continuous wood stave pipe per linear foot of various diameters from 1 to 22 ft, for untreated Douglas fir Douglas fir has a weight of approximately 36 lb per cu ft, redwood approximately 26 lb per cu ft cypress 30 lb per cu ft and longleaf yellow pine, approximately 44 lb per cu ft depending upon the degree of dryness As the specifications usually call for 8 lb of creosote oil per cu ft a proportionate increase in weight per foot is to be allowed for creosoted pipe

In the manufacture of staves, selected lumber is required, and, in addition it is most economical to use stock sizes of lumber, that is to say sizes such as 2 × 4, 3 × 4, 4 × 6, etc , so that the rejections can be used commercially for other purposes There is an economical width of stave for each diameter which in general may be approximately stated as follows

For pipes up to 24 in in diameter the width should not be greater than 4 in
For diameters of 30 in to 14 ft, the stock should be 6 in wide
For greater diameters stock not more than 8 in in width should be used

Staves as wide as 12 in have been used in some instances but are wasteful of lumber unless left with flat interior and exterior surfaces, in which case it is

impossible to get proper bearing between bands and staves and thus to prevent the development of cracks and checks.

Wood for Staves. Many varieties of timber, such as hemlock, spruce, yellow pine, Douglas fir, redwood, and cypress, have been used for the staves of wood pipe, and satisfactory pipes have been made from all these timbers. The woods most commonly used for pipes are redwood and Douglas fir, which grow in the Pacific Coast states. Spruce and yellow pine of suitable

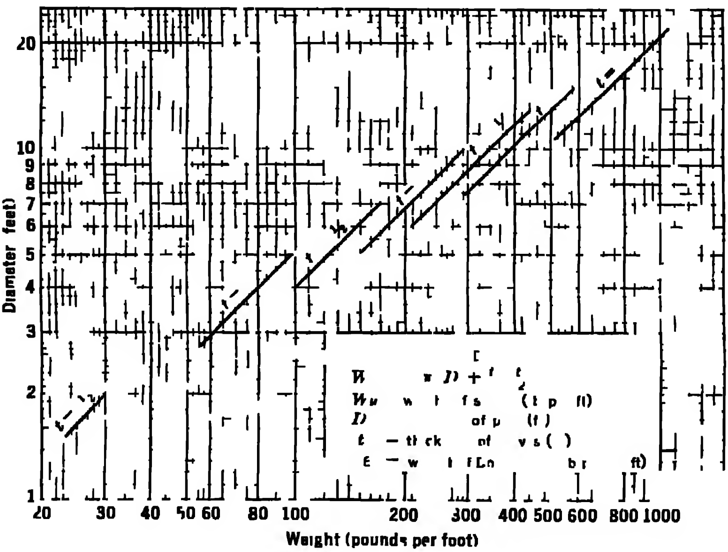


Fig. 5. Diagram showing the weight of Douglas fir staves in wood-stave pipe per foot of length. For redwood multiply by 0.72.

quality are difficult to obtain in large enough quantities; hemlock and other softwoods are becoming scarce and also contain defects which make them relatively unsuitable for long-life pipes as compared with pipes made of redwood and fir.

Although there is some dispute between the advocates of redwood and Douglas fir, the evidence seems to indicate that for untreated pipe redwood may give a somewhat longer life than fir under certain conditions of soil, climate, and use. With proper creosote or other treatment, however, it is probable that either pipe when properly designed, installed, and maintained will give satisfactory service for many years. Failures of pipe made from both woods have occurred where proper precautions in design, installation, maintenance, and use were not observed.

Wood for staves should be practically straight grained and free from shakes, and sapwood should be excluded from the exterior of the pipe. A small amount of sapwood in the interior, since it will be continuously satu-

rated, is not considered a disadvantage, especially if creosoted. No sapwood should be allowed on the exterior. A limited number of small, live knots are permissible, but should not be allowed close to ends of staves. (See specifications.)

The growth rings should preferably be nearly parallel to the circumference of the pipe. Where they are diagonal, extending through from the interior to the exterior surfaces of the stave, splitting sometimes results under high heads, and in very porous specimens of lumber excessive percolation of water may occur.

If the growth rings are practically parallel to the radii of the pipe, excessive percolation may take place through the soft rings of some woods under high heads, but this is not true of Douglas fir or redwood.

Redwood has less mechanical strength than fir or longleaf yellow pine. Lower stresses, both in bearing and bending, should be used with redwood pipe. It is to be noted, however, that the bending strength in a wood pipe is not brought into play sufficiently in properly designed installations to make this a material factor, except under extremely high heads with thin staves and bands of large cross-sectional area.

It is generally agreed that air-dried lumber is preferable for wood pipelines of redwood, which shows a considerable loss of strength when kiln-dried. Douglas fir staves, however, if kiln-dried properly, i.e. not too rapidly, show practically no loss of strength, and it is standard practice to specify kiln-dried lumber.

6. Bands. *Allowed Stresses in Bands.* Methods for determining the loading or maximum internal pressure in closed conduits, including water hammer and surges, are given in Section 3 of Chapter 31. The usually accepted working stress for steel bands is 15,000 lb per sq in. However, the working stress to be adopted depends upon the probable accuracy of the computed loading, the elastic limit of the steel used, the relative importance of the pipe, and the danger of damage to other important structures in case of failure.

Recommended values of working stresses in percentage of the elastic limit of the steel are given in Section 3 of Chapter 31.

Figure 17 shows the spacing between centers of bands from ⅝ in. to 1¼ in. in cross-sectional diameter on pipes of 1 ft to 22 ft internal diameter, for various total static heads within the limits of safe stresses in, and maximum and minimum spacing between, the bands. This diagram is based on a working stress of 15,000 lb per sq in. In using it, the sum of all elements that will simultaneously cause pressure within the pipe should be taken for the head. For instance, the sum of the static head and the head due to surge at the point under consideration should be taken, and not the static head only.

The stress in the bands, caused by swelling of the staves when first saturated with water, is treated in Section 9.

Where cradles are used, a bending moment in the bands lying in the cradle exists at the top edge of the cradle, caused by the bulging or deformation

of the pipe when filled with water. So far as the authors are aware, no determination of the magnitude of this moment has been made, and no account of it is ordinarily taken in design and construction.

Steel for Bands. Bands should be made of mild, open-hearth steel having a tensile strength of from 55,000 to 65,000 lb per sq in. and an elastic limit of not less than one half the tensile strength. Bands should be capable of being bent cold 180 degrees to a circle equal to the diameter of the band, without sign of fracture.

Threads should be U. S. Standard, and the nuts should fit snugly and yet turn easily without too much play. The bands should be threaded for a length suitable to the diameter of the pipe and sufficient to permit both easy installation and tight cinching-up of the pipe after erection.

Bands are made in one or two pieces for ease of erection. One-piece bands are ordinarily used for pipes up to 54 in. in diameter. The ordinary practice is to provide a button head on one end and a long thread at the other end, for connection to the malleable-iron shoe by means of which the bands are tightened up. This construction permits the button head to be underneath the threaded end in the shoe, thereby allowing the bands to be spaced as closely together under maximum head conditions as is possible.

A less frequently used construction for moderate heads is made by threading both ends of the band. This necessitates having the threaded ends pass one another in the shoe, thereby considerably increasing the minimum spacing between the bands.

With two-piece bands, the usual construction is to provide the lower halves of the bands with threaded ends and the upper halves with button heads on which the shoes are hung while installing the bands.

The threads should be so designed and made that the tensile strength at the root of the threads is equal to that of the body of the bands. Rolling the threads will accomplish this result, and such bands are so guaranteed by the manufacturers. If cut threads are used, the net strength of the band will be that of the area at the root of the thread, requiring larger bands. Upsetting the ends to a diameter ⅛ in. greater than that of the body of the rod will give an area at the root of the thread approximately equal to that of the rod, and permit stressing the rod to its efficient value. If this is done, however, wider shoes will be required to fit the upset ends, resulting in a wider minimum spacing of bands and a lower maximum head under which the pipe may be used. The nuts used are somewhat thicker than standard nuts, so as to avoid the possibility of stripping the threads. Plate-steel washers are used between the nuts and the shoes.

Maximum and Minimum Spacing of Bands. The maximum spacing between bands is determined by the length of unsupported stave or of the joint between staves which will not leak under the particular head under which the pipe is used. In present-day practice, this maximum spacing is usually from 6 to 10 or even 12 in., in some extreme cases, and depends upon the thickness of the stave as well as the pressure head. For the thicknesses

shown in Fig. 4 and the band spacings as in Fig. 17, the bending moments in the staves between the adjacent bands are well within safe limits. The deflection may, however, be great enough to cause small leaks between staves, if bands of too large cross-sectional area, and consequently widely spaced apart, are used with relatively thin staves. Within the limits of stave thicknesses and sizes and spacing of bands shown in Figs. 4 and 17, excessive deflections in staves will not occur.

Extra bands are required to support the ends of staves at the joints when the band spacing exceeds 6 in.

The minimum spacing between bands is obtained by staggering the shoes around the circumference of the pipe, usually in groups of three, with one shoe at approximately the horizontal diameter, one above, and one below, as indicated in Fig. 3. This arrangement permits the bodies of two bands on either side to be brought into contact with a shoe, so that half the width of the shoe plus half the diameter of the band used determines the minimum spacing and the maximum head under which the pipe may be used.

Figure 6 shows the minimum spacing from center to center of various sizes of bands from $\frac{3}{8}$ to $1\frac{1}{4}$ in. in diameter, with shoes of conventional design and for bands having button heads.

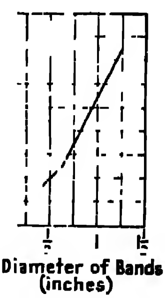


Fig. 6. Diagram showing the minimum spacing of bands for wood-stave pipelines.

It is customary to vary the spacing of the bands for small increments of pressure head (usually taken at 5 ft or less), and thus to make the metal in the bands work at practically its full efficiency. In this particular, wood pipe has an advantage over pipe in which the steel shell or reinforcement increases by large percentage increments or must be of a certain minimum thickness for the sake of stability, and hence cannot be as economically employed as the steel used in wood-pipe bands. This advantage is greatest for large pipes at low heads.

For machine-banded pipe, the bands may take either the form of round steel wire, or of flat steel galvanized bands, as may be specified or preferred.

Diameter of Bands. The maximum cross-sectional diameter of band is largely a matter of convenience in erection. For pipe of, say, 12-ft diameter, a band $\frac{7}{8}$ in. in diameter is capable of being applied without too much labor in erection. Bands having diameters of 1 in. to $1\frac{1}{4}$ in. can be used if required by the static head, but the labor of erection is of course increased.

The minimum diameter of bands varies with size of pipe, and allowance must be made for the possibility of corrosion and reduction of area, which is, of course, a more serious matter with the smaller than with the larger bands. In practice, $\frac{1}{2}$-in. diameter bands are about as small as should be used for continuous pipe of 5 ft or more in diameter. Figure 17 shows the band spacing for wood-stave pipe for different sizes of bands to resist different heads. The head used should be the total static head plus the possible surge

of the pipe when filled with water. So far as the authors are aware, no determination of the magnitude of this moment has been made, and no account of it is ordinarily taken in design and construction.

Steel for Bands. Bands should be made of mild, open-hearth steel having a tensile strength of from 55,000 to 65,000 lb per sq in. and an elastic limit of not less than one half the tensile strength. Bands should be capable of being bent cold 180 degrees to a circle equal to the diameter of the band, without sign of fracture.

Threads should be U. S. Standard, and the nuts should fit snugly and yet turn easily without too much play. The bands should be threaded for a length suitable to the diameter of the pipe and sufficient to permit both easy installation and tight cinching-up of the pipe after erection.

Bands are made in one or two pieces for ease of erection. One-piece bands are ordinarily used for pipes up to 54 in in diameter. The ordinary practice is to provide a button head on one end and a long thread at the other end, for connection to the malleable-iron shoe by means of which the bands are tightened up. This construction permits the button head to be underneath the threaded end in the shoe, thereby allowing the bands to be spaced as closely together under maximum head conditions as is possible.

A less frequently used construction for moderate heads is made by threading both ends of the band This necessitates having the threaded ends pass one another in the shoe, thereby considerably increasing the minimum spacing between the bands

With two-piece bands, the usual construction is to provide the lower halves of the bands with threaded ends and the upper halves with button heads on which the shoes are hung while installing the bands.

The threads should be so designed and made that the tensile strength at the root of the threads is equal to that of the body of the bands. Rolling the threads will accomplish this result, and such bands are so guaranteed by the manufacturers. If cut threads are used, the net strength of the band will be that of the area at the root of the thread, requiring larger bands. Upsetting the ends to a diameter ⅛ in. greater than that of the body of the rod will give an area at the root of the thread approximately equal to that of the rod, and permit stressing the rod to its efficient value. If this is done, however, wider shoes will be required to fit the upset ends, resulting in a wider minimum spacing of bands and a lower maximum head under which the pipe may be used. The nuts used are somewhat thicker than standard nuts, so as to avoid the possibility of stripping the threads. Plate-steel washers are used between the nuts and the shoes.

Maximum and Minimum Spacing of Bands The maximum spacing between bands is determined by the length of unsupported stave or of the joint between staves which will not leak under the particular head under which the pipe is used In present-day practice, this maximum spacing is usually from 6 to 10 or even 12 in., in some extreme cases, and depends upon the thickness of the stave as well as the pressure head. For the thicknesses

shown in Fig. 4 and the band spacings as in Fig. 17, the bending moments in the staves between the adjacent bands are well within safe limits. The deflection may, however, be great enough to cause small leaks between staves, if bands of too large cross-sectional area, and consequently widely spaced apart, are used with relatively thin staves. Within the limits of stave thicknesses and sizes and spacing of bands shown in Figs. 4 and 17, excessive deflections in staves will not occur.

Extra bands are required to support the ends of staves at the joints when the band spacing exceeds 6 in.

The minimum spacing between bands is obtained by staggering the shoes around the circumference of the pipe, usually in groups of three, with one shoe at approximately the horizontal diameter, one above, and one below, as indicated in Fig. 3. This arrangement permits the bodies of two bands on either side to be brought into contact with a shoe, so that half the width of the shoe plus half the diameter of the band used determines the minimum spacing and the maximum head under which the pipe may be used.

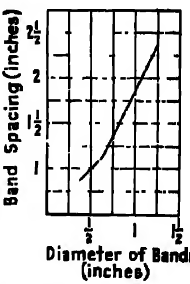


FIG. 6. Diagram showing the minimum spacing of bands for wood-stave pipelines.

Figure 6 shows the minimum spacing from center to center of various sizes of bands from ⅜ to 1¼ in. in diameter, with shoes of conventional design and for bands having button heads.

It is customary to vary the spacing of the bands for small increments of pressure head (usually taken at 5 ft or less), and thus to make the metal in the bands work at practically its full efficiency. In this particular, wood pipe has an advantage over pipe in which the steel shell or reinforcement increases by large percentage increments or must be of a certain minimum thickness for the sake of stability, and hence cannot be as economically employed as the steel used in wood-pipe bands. This advantage is greatest for large pipes at low heads.

For machine-banded pipe, the bands may take either the form of round steel wire, or of flat steel galvanized bands, as may be specified or preferred.

Diameter of Bands. The maximum cross-sectional diameter of band is largely a matter of convenience in erection. For pipe of, say, 12-ft diameter, a band ⅞ in. in diameter is capable of being applied without too much labor in erection. Bands having diameters of 1 in. to 1¼ in. can be used if required by the static head, but the labor of erection is of course increased.

The minimum diameter of bands varies with size of pipe, and allowance must be made for the possibility of corrosion and reduction of area, which is, of course, a more serious matter with the smaller than with the larger bands. In practice, ½-in. diameter bands are about as small as should be used for continuous pipe of 5 ft or more in diameter. Figure 17 shows the band spacing for wood-stave pipe for different sizes of bands to resist different heads. The head used should be the total static head plus the possible surge

head at a given point. Figure 7 shows the weight of hardware, including bands, shoes, etc., per linear foot of pipe for different pipe diameters, heads, and sizes of bands.

Bearing on Staves. Bands should be designed so as not to indent the wood over an area greater than 90 degrees of the circumference of the band. For Douglas fir, the bearing value of bands and their connecting shoes should not

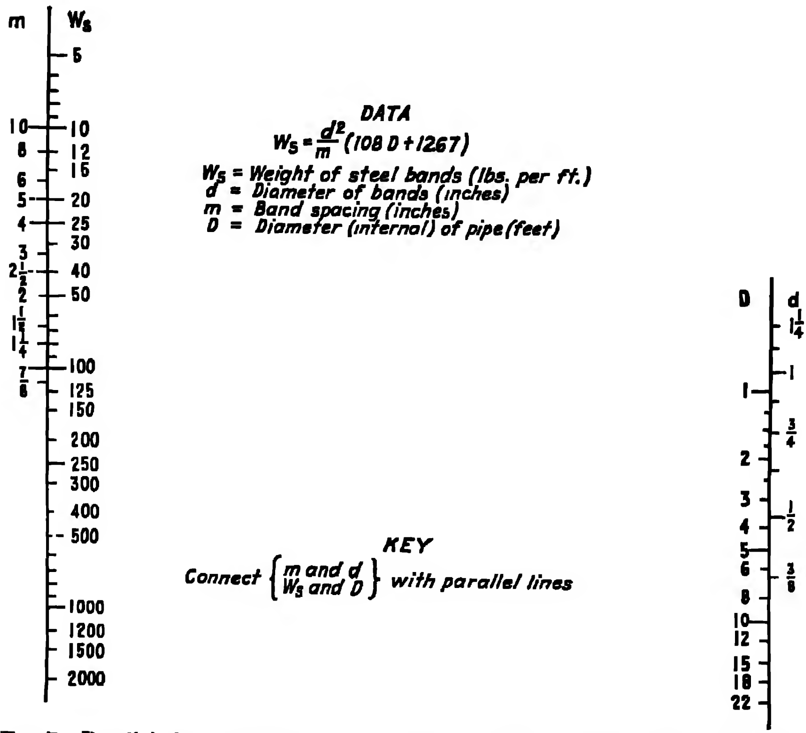


FIG. 7. Parallel chart showing the approximate weight of steel bands, shoes, and hardware in continuous wood-stave pipe per foot of length of pipe.

exceed 800 lb per sq in. of contact surface; for redwood, this should be reduced to 600 lb per sq in.

Such bearing pressures are due to (*a*) the pressure head of water within the pipe, including surges, pressure due to swelling of wood, etc.; and (*b*) the weight of the pipe itself and the contained water.

These two bearing pressures are combined in a pipe supported on one or more bands lying in a cradle. Hence a sufficient number of bands must be provided in each cradle to keep the maximum bearing pressure within the above limits, unless other means, such as grouting between cradle and pipe shell, are resorted to for support, in which event only the pressure due to pressure head will come into play.

In cradles, the pressure due to weight of pipe and water is supported on the horizontal projected length of band lying in the cradle and is a maximum at the center of the cradle; the component due to pressure head, etc., is proportional to the equivalent pressure head above any point on the circumference.

7. Tongues or Butt Joints at Ends of Staves. The ends of adjacent staves are made tight against leakage by means of tongues or butt joints. The simplest form of the tongue type is a rolled-steel plate of No. 10 B. W. gage, Fig. 8. These tongues are cut square and true and are usually made 1¾ in. wide and of a length 1/16 to ⅛ in. greater than the width of the stave at the slot. The slots or saw cuts in the ends of the staves into which the tongues are inserted are usually made ¾ in. deep, allowing the tongues to cut ⅛ in. into the wood.

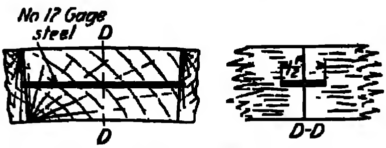


Fig. 8. Tongue-type butt joint.

In freezing climates the butt ends of staves, especially near the top of the circumference, often split outward. Apparently the splitting is caused by the alternate freezing and thawing of water which accumulates in the small pockets formed by the spaces between stave ends and the metal tongues. This has been overcome by placing galvanized sheet metal over the joints and under the adjacent bands with a plastic bituminous material underneath. It is quite possible that the plastic material alone might be sufficient, provided the pockets are carefully filled with it. A device similar to that described in the following paragraph is also effective

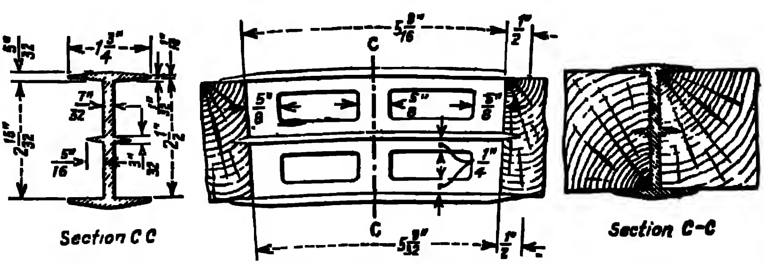


Fig. 9. Kelsey malleable-iron butt joint.

Figure 9 shows the Kelsey butt joint, which makes a stronger joint than the simple tongue and is particularly applicable when pipe is installed to work under conditions of suction or vacuum, as it transmits the stresses from the joint to all four of the adjacent staves, and aids in keeping the ends of staves from splitting.

In erecting pipe, when entering the tongues or shoes at the end joints, the adjacent staves should be spread apart by means of chisels so as not to score the radial edges of staves unnecessarily.

8. Shoes for Bands. Shoes should be made of high-class, malleable cast iron, sound, smooth, and free from defects.

The shoes that connect the ends of bands should be so designed as to have a strength at least equal to that of the gross cross-sectional area of the bands themselves. Several designs of shoes have been made and used satisfactorily, but by careful attention to design, it has been possible to reduce greatly the weight of metal required in order to meet this specification. Figure 10 shows

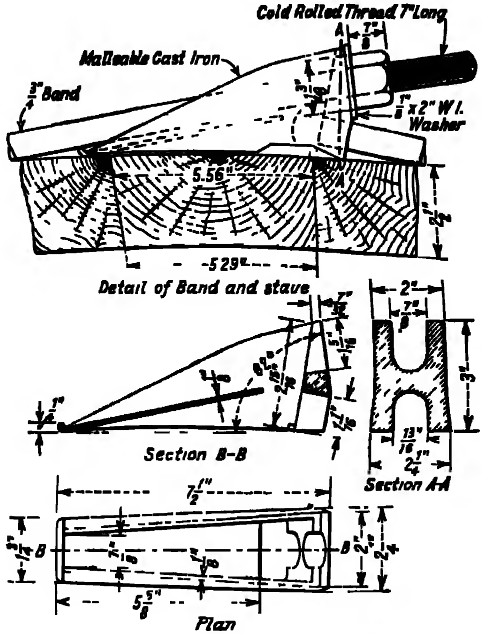


FIG. 10. Typical band shoe

a satisfactory type of shoe. In the usual designs, provision has been made for bands having a button head at each end of the top half of the band and threads on the lower half. The design most frequently used has open-sided slots, which allow the shoe to be slipped under the button head, and the other part of the band, with the nut partly screwed on, to be slipped into place without having to slide the shoe endwise over either section of the band. Sufficient bearing area should be provided to keep the bearing on the wood within the limits specified in Section 6.

Some of the older designs were so made that the shoes had to be slipped longitudinally over the bands; and, in certain of them, the bands were provided with threads and nuts for both top and bottom sections.

9. Allowance for Initial Swelling and Compression of Wood. In addition to resisting the static head of water at any point, the bands must

resist any additional head due to possible surges and also, at least temporarily, the pressure produced by the initial swelling of the staves when first saturated on filling the pipe. The swelling pressure to be resisted by a single band is that on an area having a width equal to the radial thickness of the staves and a length equal to the longitudinal spacing, center to center, of bands. Adams* concludes that the ultimate pressure due to swelling may be taken safely at 90 to 100 lb per sq in. In the experiments with both red-

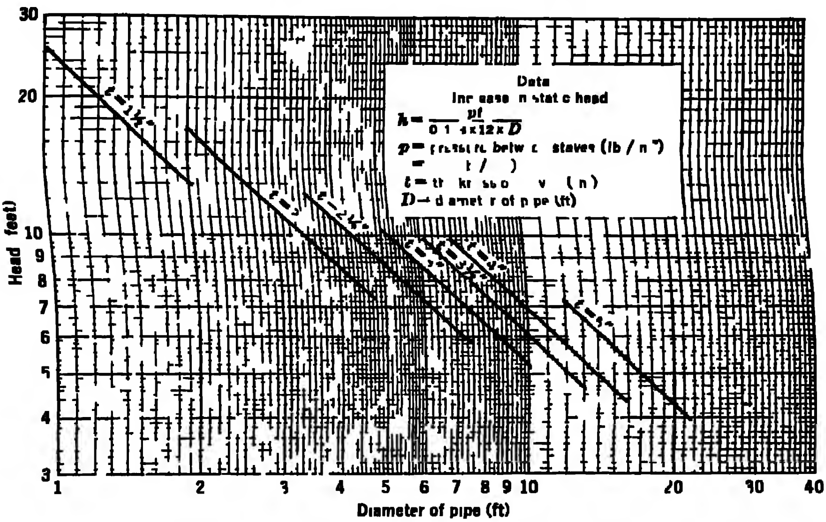


FIG. 11. Diagram showing the equivalent increase in static head on wood-stave pipe caused by initial swelling of wood on filling pipe with water. For 10-in band spacing.

wood and Oregon fir quoted by Adams and in the discussion of his paper by D. C. Henny, initial swelling pressures up to [illegible]00 lb per sq in were observed, but these soon decreased as the wood became saturated to values between 75 and 150 lb per sq in. Figure 11 shows the static heads equivalent to a pressure of 90 lb per sq in for the usual thicknesses of staves and diameters of pipe, which was taken by Adams as the most probable value as a result of the experiments. The values shown in Fig. 11 are based upon 10-in spacing between centers of bands. After the wood has once become thoroughly saturated and the bands have indented themselves into the staves, this pressure is largely relieved.

From the foregoing it is evident that the band stress resulting from the swelling of the wood is of greater importance where bands are widely spaced. As the spacing is reduced, this stress becomes a smaller portion of the total stress which the bands must resist. Since the initial swelling pressure de-

*See p. 37 of Ref. 1.

creases rapidly, it is not necessary to take the excess over the ultimate pressure into account further than to keep the initial band stress below the elastic limit of the steel. Some of the wood-pipe manufacturers disregard the swelling pressure entirely in designing, on the assumption that it practically disappears within a few weeks after the pipe is put in service.

When the initial stress due to swelling is high, it is advisable to fill the pipe slowly, thereby affording an opportunity for the swelling pressure to decrease before subjecting the pipe to the full static head.

10. Minimum Allowed Radius. Continuous wood pipe has a certain degree of flexibility and may be laid to any desired radius not less than approximately fifty times the normal diameter of the pipe. Somewhat sharper curves can be used but will cause difficulty in erection. Horizontal curves are formed by pulling the pipe into line by means of chain blocks after a few bands are in place. This becomes very difficult if too small radii are attempted.

11. Pipe Supports. Some form of exterior support or saddle, extending at least partially up the sides of continuous wood-stave pipe, is usually necessary. These supports range in form from (1) a simple cross sill of timber, with additional blocks of wood on either side, which are cut to the radius of the outer circumference of the band; to (2) a concrete, masonry, cast-iron, or structural-steel saddle extending up toward or above the horizontal diameter of the pipe, formed to the external radius of the bands, or, at least, supporting the pipe at three or more points. Typical saddles, which are also suitable for steel pipe, are given in Figs. 6 of Chapter 31.

The loading imposed on the cradles by the weight of the pipe itself, plus the pressure within, and the stresses caused thereby, is determined in the same manner as for steel pipe. The reader is referred to Section 13, Chapter 31, for loadings for steel pipe.

The current practice as to spacing and width of cradles longitudinally of the pipeline is shown in Figs. 12 and 13, in which the maximum and recommended values are graphically shown. The area of contact between the cradle and the pipe should not be smaller than that required to give a bearing pressure between the bands and the wood of 800 lb per sq in. for fir and pine, and 600 lb per sq in. for redwood. Carrying the cradle up to or above the horizontal diameter of the pipe greatly increases its ability to withstand external loading, crushing, or vacuum.

Figure 12 shows the *minimum* and the recommended included angle or portion of the circumference of a wood-stave pipe which should be supported in the cradle, as well as the best width of the seat of the cradle in which the pipe lies. Inspection shows that, for all but small pipes (3 ft or less in diameter), some bulging occurs immediately above the top of the cradle even though it extends up to the horizontal center line of the pipe; and that the first fully exposed stave above the cradle is usually forced outward beyond the stave below it. This may be prevented and a stronger and more satisfactory pipe insured by extending all cradles at least 5 degrees above the horizontal

center line, i.e., making the included angle at least 190 degrees, or greater, as shown in Fig. 12.

From the above, it is apparent that, as the diameter of pipe increases, so also should the thickness of the cradle and the angle of contact.

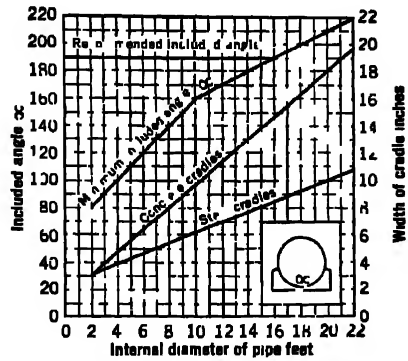


Fig. 12. Recommended width of cradle seats and included angle for wood-stave pipes.

Where the spacing between cradles is too great, the pipe flattens at the cradles and sags between them.

Particular attention should be given to foundation conditions to insure that the saddles will not settle or get too far out of line. In this particular,

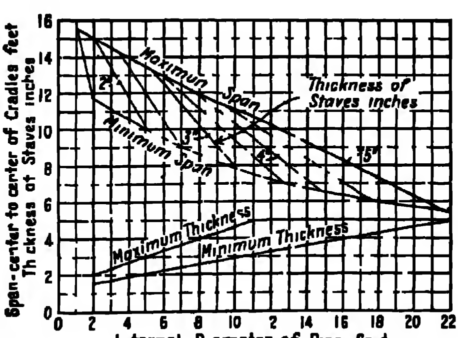


Fig. 13. Spans and thickness of staves for wood-stave pipe.

wood pipe possesses an advantage over other types of pipe in that it is flexible and can accommodate itself to slight changes of grade and alinement without being seriously damaged or having its usefulness impaired.

12. Painting and Creosoting. The most common method of treatment is by the pressure-vacuum creosoting process, which leaves a residue of about

8 lb of creosote oil per cu ft of wood. This treatment will be effective over varying periods of time, depending upon the climatic conditions and the character of the wood. Such treatment can be renewed by a brush or air-sprayed coat or coats of light creosote oil, as conditions may require.

Staves tend to decay at the ends and where they are bruised. It is therefore desirable to dip the ends in creosote oil or other preservative before laying in the pipe. Creosote oils appear to be the most suitable for the purpose as they are not soluble in water and they inhibit rot and fungi.

Experiments made with Douglas fir creosoted by the high-temperature process show a loss of strength up to 30%. Therefore, the low-temperature process should be used for staves.

The life of a pipe constructed with untreated staves may be lengthened by spraying or painting its exterior with hot light creosote oil, especially before the pipe has been filled with water. Later, after the more volatile oils have evaporated, it should be given a hot sealing coat of refined asphaltum, if the longest possible life is desired. Before applying this treatment to pipes filled with water, obviously all leaks must be carefully stopped. Such treatment must be repeated at more or less frequent intervals if best results are to be obtained. Some question appears to exist whether there is an interaction between creosote oil and asphalt-base paints. Either tar or asphalt paints will serve as efficient protection to the steel and metal parts of the pipe.

For greatest permanence, wood-stave pipes should be built of creosoted staves, regardless of the pressures to which they are to be subjected or the conditions under which they are to be laid or used. A hot coat of refined asphaltum applied to a pre-creosoted pipe after erection will add to its useful life. Pipes of untreated wood cannot be expected to have as long life as those whose wood and metal parts are properly treated and maintained. A recent inspection of wood-stave pipes in the eastern United States showed that those pipes which were in the best condition for their age were those which had been treated as described above.

Ordinary paints have been applied to wood pipe, but the objection appears to be that they require more frequent renewal in order to preserve the pipe satisfactorily.

When repairs or replacements become necessary, it is sometimes found that, if not protected, the threads on the bands have become so rusted that it is impossible to get the nuts off. It is then necessary to cut the bands, and the loss of the bands and nuts results. Initial painting and occasional repainting of bands and threads with asphalt- or tar-base paint will obviate this difficulty.

13. Life of Wood-stave Pipe. The life of wood-stave pipe has been the subject of much discussion; in general, it may be said to depend upon:

(*a*) Its design and the method of installation, whether exposed, partially buried, or buried.

(*b*) The nature of the soil with which it comes in contact.

(c) The protection afforded to the staves and metal appurtenances, i.e., whether treated or untreated with paint or other preservative, galvanized, etc.

(d) Maintenance after installation.

(e) Whether the pipe is continuously filled with water or intermittently filled and emptied and allowed to dry out.

(f) Climatic conditions.

(g) The wood of which it is composed.

The life of wood-stave pipe, under favorable conditions with proper treatment, may be assumed to be 20 to 30 years. Experience with a number of 3- to 12-ft untreated wood-stave pipes in New York state indicates a useful life in excess of 30 years for pipes carefully maintained. Pipes sprayed with creosote as described above give every indication of a much longer life, while precreosoted pipes show little evidence of deterioration.

If a careful and intelligent study and appraisal of the conditions affecting the life of a given installation is made, suitable designs, materials and protection provided, and the pipe and related structures properly inspected and maintained, a useful life of 30 to 50 years or more should be attained for a pipe continuously filled with water, except under unfavorable conditions which will be more fully discussed in the paragraphs following.

In general, it may be stated that a wood-stave pipe of fir should always receive preservative treatment unless a life of 5 to 25 years, depending on the surrounding conditions, is all that is required. As is elsewhere pointed out, it is a great advantage to have the entire pipe exposed so that it may be readily inspected and the preservative treatment renewed when necessary. If this is done, the maximum useful life may be attained.

(a) *Design and Installation* When a pipe is to be used temporarily, the question of the manner of installation is not very important. If, however, long life is desirable or essential, experience indicates that the maintenance of pipe in good condition is much facilitated by installing it entirely in the open on suitable cradles. A pipe so installed can be easily and frequently inspected so that any repairs or maintenance can be quickly and easily accomplished before serious injury or damage results.

Wood-stave pipe has been laid underground but such practice is not recommended for the following reasons: when buried, the wood decays more or less rapidly (very rapidly in some soils); small leaks may go unnoticed for long periods of time, while those which are found may manifest themselves long distances downgrade from their sources; exterior inspection is impossible, and maintenance is slow and costly, because the pipe must be exposed by excavation before its condition can be ascertained and repairs made.

Sufficient space should be left under and around a pipe laid aboveground to insure, so far as possible, that no earth or other material can find its way into the trench and remain in contact with the pipe; or, at least, means should be provided for removing such material soon after it is deposited.

For the larger sizes of pipe, some form of support or cradle at frequent intervals is very desirable and, in fact, practically essential. No positive

formula has been worked out, but in practice it is customary to space such supports as described in Section 11, "Pipe Supports." There is an instance on record of a large wood pipe remaining intact and filled with water after the support had been washed out from under a section 50 ft long. Had the pipe been emptied and allowed to dry out, this section would undoubtedly have collapsed.

Wood pipe for use where suction or vacuum may be produced, as in condenser lines, should be designed with thicker staves, and some device such as the Kelsey butt joint should be used between ends of staves to distribute the stresses. If the pipe is large and the vacuum high, the safety of the pipe may require that it be incased in concrete.

Wood-stave pipe is not suitable for pump suction lines on account of the practical impossibility of preventing air leakage through small leaks in the pipe, unless the pipe is incased in concrete or other material which will effectually prevent such leakage.

The design and installation of a satisfactory wood-stave pipe involves special knowledge, obtainable only from actual experience. It will usually be found advisable to intrust the construction to one of the established manufacturers and to consult with their engineers about design and layout, thus taking advantage of their experience, which is bound to be much more extensive than that of most designing and construction engineers.

(*b*) *Protection: Painting and Treatment of Staves.* Many pipes have been laid without any treatment of the staves. The useful life of wood depends upon its ability to resist decay and the attacks of fungi. Hence, treatment with a preservative which inhibits such attacks and renewal of the treatment at suitable intervals will manifestly greatly lengthen its life. Such treatment is described in Section 12.

Where maximum permanence is desired, such treatment is essential. An untreated pipe cannot have as long a life as one that is properly treated and maintained.

(*c*) *Maintenance.* A pipe should be inspected at sufficiently short intervals to insure that no serious deterioration has taken place since the previous inspection. Any serious leaks should be stopped up by the means suggested in Sections 4 and 18, and the protective coating or treatment should be renewed whenever inspection reveals that this is necessary. Also, all structures and appurtenances of the pipeline should be renewed or repaired whenever any sign of failure or serious damage is noticed. If these precautions are taken, the life of any pipeline is bound to be greatly extended.

(*d*) *Pipe Continuously or Intermittently Filled.* A pipeline that is continuously filled with water will not deteriorate as much as one that is emptied and allowed to dry out for longer or shorter intervals of time.

(*e*) *Climatic Conditions.* In extremely hot, dry climates, as in the arid regions of the western United States, excessive checking of the staves occurs. In climates where hot, dry periods alternate with wet periods, allowing the exterior of the wood to become alternately saturated and dry, decay is

hastened. Preservative treatment will greatly minimize both these conditions and prolong the useful life of the pipe.

(f) Wood for the Staves. The kind of timber used for the staves has a marked effect on the life of the pipe, as explained in Section 5.

A report of an investigation by the U. S. Reclamation Service, quoted from an article by W. H. Nalder and furnished by the Denver office of the Service, has resulted in the following conclusions as to conditions tending to shorten the useful life of wood pipes:

(*a*) Intermittent service. Pipes full of water during the irrigation season and emptied during the nonirrigation season.

(*b*) Installations made in gravelly or open soils.

(*c*) Installations made in alkali soil, resulting in destruction of the joints and wire winding.

(*d*) Installations of machine-banded pipe in which separate wooden collars are used at joints, and in which these collars tend to rot out while the pipe walls remain in good condition, due evidently to the fact that the collars are not so thoroughly saturated in service as the walls of the pipe.

(*e*) Installations under low hydrostatic head, say, less than 15 ft, which is not sufficient to saturate uncreosoted wood properly.

14. Expansion Joints not Necessary. Owing to the low coefficient of expansion of wood, the relatively short lengths of staves, and the fact that the end joints are made tight by tongues or butt joints, which permit a slight movement without causing perceptible leakage, expansion joints in wood pipe are unnecessary except at connections to other structures or other types of pipe.

15. Freezing. No record of serious effects in wood-stave pipe of the sizes and lengths used in hydroelectric installations is available. An account of freezing in a 26½-mile section of municipal water-supply pipe in the state of Washington was given in *Engineering News-Record* of December 31, 1925, page 1077. Eight miles of this approximately 28-in.-diameter line was left exposed, resulting in the formation of some ice each winter. In the winter of 1924 and 1925, a 10-in. thickness of ice formed, reducing the interior diameter of pipe to 8 in. This ice was subsequently removed at a sandbox in the line when the weather became warm enough to permit the ice to thaw.

In this particular case, it was found that freezing caused spalling or breaking off of the outer half of the ends of the staves over the steel-plate butt joints. These breaks were repaired by placing galvanized-iron plates over each break and fastening them on by means of two additional bands, quite similar to the repair mentioned in Section 18. (See also Section 7 for other suggestions.) A device similar to the Kelsey malleable-iron butt joint (Fig. 9) would prevent or greatly reduce splitting at ends of staves.

In freezing climates, snow invariably collects to greater or less depths on the top of wood pipe and undoubtedly helps greatly to insulate the pipe against freezing. No cases of any serious loss of head in wood pipe during freezing weather have come to the attention of the writer. It is probable that very little ice forms inside such pipes, except perhaps where a pipe is laid up and water is permitted to stand in it for a considerable length of time.

16. Accessories. *Connections to Steel Pipe.* Wood pipe should be connected to steel pipe by means of a slip joint, on account of the possibility of expansion and contraction of the steel pipe. Figure 14 shows a typical slip-joint connection which has been satisfactorily used. A joint should have a retaining ring at the inner end, in front of which oakum or hemp packing is tightly driven up and, in the simplest possible case, held in place by a retaining ring as indicated.

Under heavy pressures such a joint will leak more or less. If leakage is objectionable, it will be necessary to install a ring of Z section, which is capable

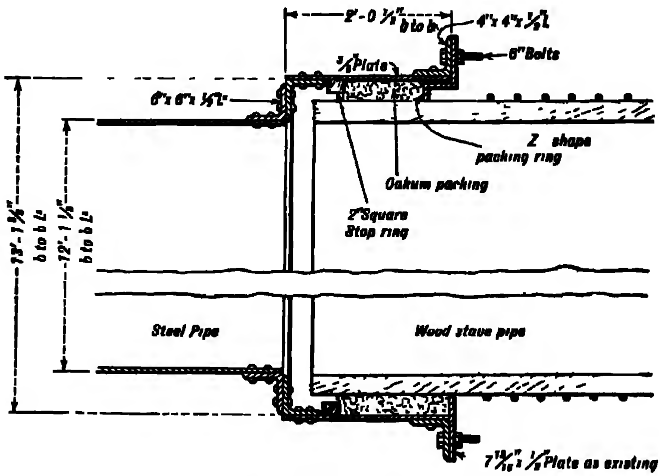


FIG. 14. Detail of Trenton Falls expansion joint at junction of steel and wood-stave pipelines. (Central New York Power Corporation)

of some movement and can be tightened up by means of nuts on the studs shown in the figure. Where a joint made as above described cannot be satisfactorily kept tight, lead wool may be tightly calked in for the last 2 in. or more at the outer end of the joint, after which the retaining ring is to be put on and tightened up. Another method is to pack the joint tightly with oakum at the inner end, then pack tightly with dry cement mortar to within 3 in. from the outer end, and the remainder with lead wool tightly calked, and then tighten up the retaining ring.

The annular space outside the wood varies from 1 to 3 in. in width in different designs. The smaller the space in which the oakum or other packing can be tightly calked, the better are the results obtained.

At such joints, the steel pipe should, if possible, be securely anchored in a concrete block; otherwise, appreciable movement and leakage may take place.

Connections to Concrete Pipe, Dams, etc. For connecting wood pipe to concrete pipe or other concrete work, as at dams, etc., an annular joint, quite similar to that just described for steel pipe, should be provided. This joint

will not differ essentially from that described above, except that, where connection is made into a large mass of concrete, the retaining ring may not be required, particularly under low heads. If the retaining ring is omitted, the outer portion of the packing should preferably be of lead wool solidly calked in.

Connections for Surge Tanks, Vents, Manholes, Drains, etc. A typical vent pipe connection is shown in Fig. 15. Connections for manholes, drains,

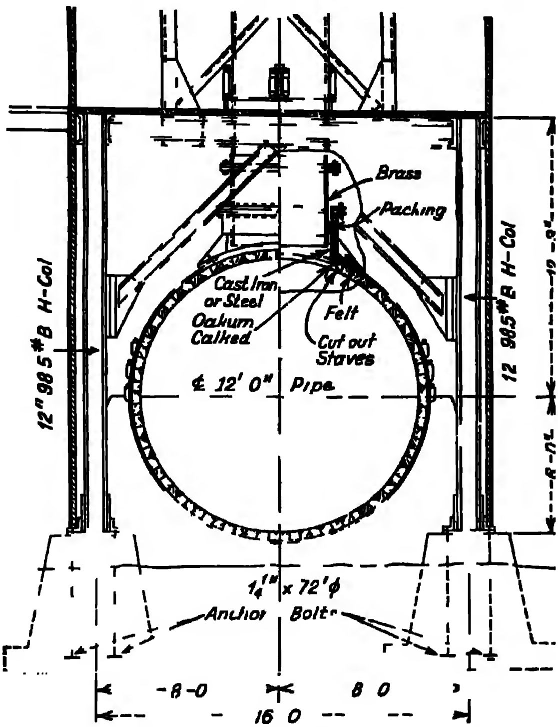


Fig. 15. Typical connection of steel vent pipe.

or other openings of less diameter than the normal diameter of the pipe are most readily made by means of castings which are generally of the form suggested in Figs. 15 and 16 of this chapter and Fig. 6 of Chapter 28.

The essential features of such connections are a metal pad, usually of cast iron but preferably of steel for high pressures, fitted to the outer circumference of the staves and provided with lugs corresponding to the shoes used for connecting the bands. In addition, a hole which closely fits a lip formed on the inner side of the casting should be sawed in the staves. Watertightness may be secured by means of sheets of felt saturated with asphalt and laid between the casting and the stave. Calking between the projecting lip and the wood with oakum saturated with asphalt also aids in making such a joint watertight.

Sections of bands are cut off and threaded to suit the lugs on the sides of the casting, and are made of lengths which will secure the proper staggering of the shoes at the sides of the pipe. For tall and heavy vent pipes, etc., it is necessary so to arrange the design, as indicated in Fig. 15, that the vent pipe itself is supported exterior to the pipe and the slip joint provided with a packing gland, so that the "breathing" of the pipe can take place independently of, and without causing movement of, the vent pipe or surge tank. The inner member of the slip joint should preferably be made of brass so as to have no tendency to rust fast to the iron or steel casting attached to the pipe.

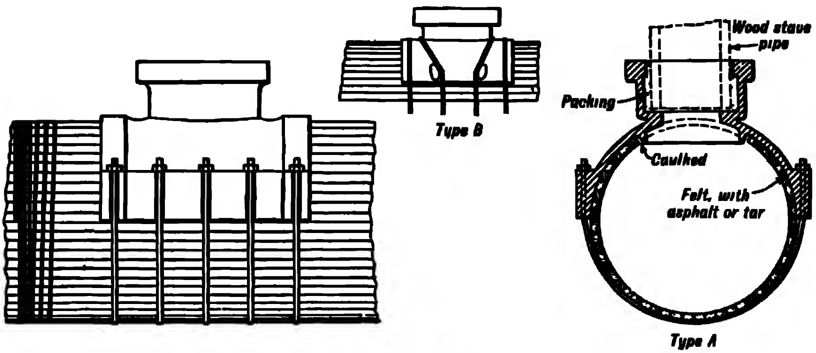


FIG. 16. Typical branch connection for wood-stave pipe

Special Bends. Where the radius of curvature is smaller than that to which the wood pipe can be successfully bent, it is necessary to install special bends which may be made of plate steel, cast iron, or concrete. These bends should be provided with joints as described above, under "Connections to Concrete Pipe, Dams, etc." and should be securely anchored against movement.

17. Special Requirements and Precautions. In laying out wood pipe, it is desirable to provide sufficient space between adjacent pipes and between pipes and other structures, such as retaining walls, piers, etc., to permit ratchet wrenches to be operated in tightening the nuts or cinching the pipe, and also to permit workmen to go completely around the pipe, for the purpose of erection, inspection, etc. The minimum space for this purpose is about 18 in.

In laying out the pipe, it is necessary to keep the profile below the lowest possible hydraulic gradient under extreme operating conditions. If, as the result of some temporary condition such as unexpectedly low head, or pond elevation, any points fall above such gradient, adequate vents, air valves, or vacuum valves should be provided. Such vents and valves should be completely protected against freezing, as otherwise they might be inoperative when required to function, and in several instances, their freezing has resulted in collapse.

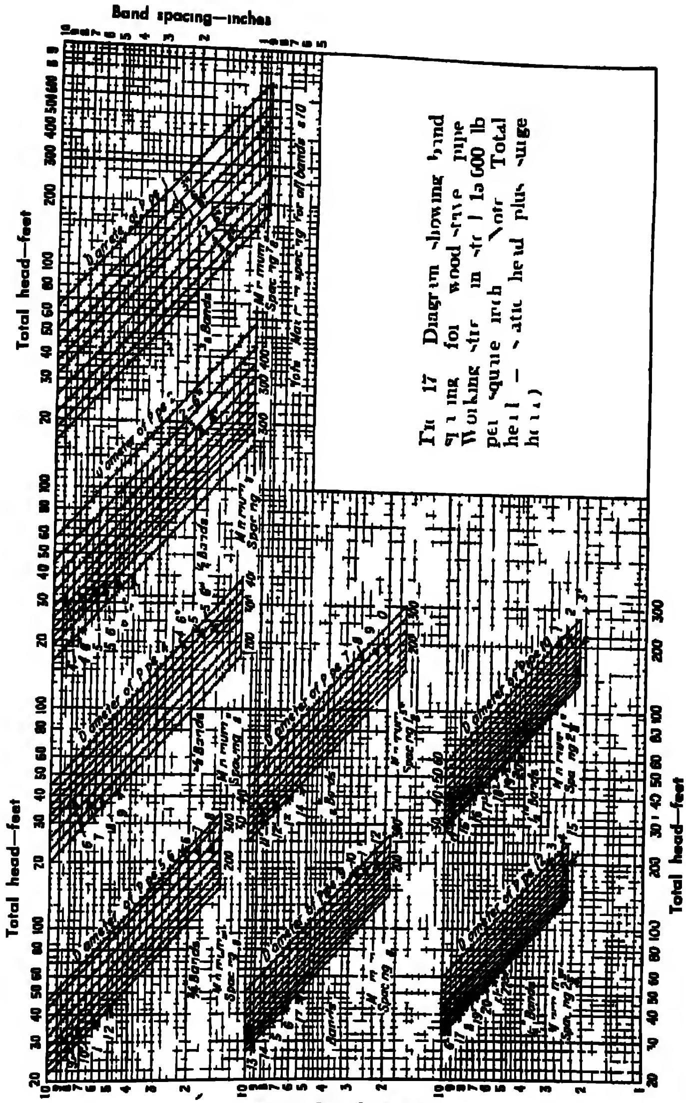


Fig 17 Diagram showing band spacing for wood-stave pipe Working stress in steel 15,600 lb per square inch (Note Total head = static head plus surge head)

Every precaution should be taken to insure the safety of a pipeline against damage or failure due to washouts from any cause. Wherever possible with economy, foundations for supporting structures should always be carried to rock or other firm bearing strata. Where streams flow beside or underneath a pipe, suitable retaining walls or other construction should be provided to limit the possibility of washout in cases of flood.

In sandy or easily eroded soils, a very satisfactory and efficient precaution against serious scour was provided in at least one case by driving rows of short sheet piling at right angles to the center line of the trench and carrying them a short distance up the sides of the trench, so that any conceivable volume of water from ordinary leakage would be compelled to flow over the top of the successive rows of sheeting. These rows were spaced sufficiently close together so that even on rather steep grades there was but little vertical fall between successive rows. Where the fall was considerable, the length of sheeting was made at least twice the vertical height between successive rows.

On fills, unless they are of extremely porous material, not easily eroded, it is important that means be provided for controlling the flow of water resulting from leaks, and for carrying it to and down paved or otherwise protected channels to a safe distance from the pipeline. Unless this is done, there is a possibility of a washout due to leaks which occur when the pipeline is first filled and may also occur at some subsequent time. A pipe should also be protected from the possibility of being floated by the presence of water around it at a time when it is empty.

18. Maintenance. One occasionally finds leaks that cannot be closed by the customary plugs and wedges. In several instances when this has occurred, steel plates of approximately No. 10 gage, rolled to the radius of the outer circumference of the pipe, have been inserted under the partly loosened bands, after which the bands were again tightened, with satisfactory results.

A repair was made to a large pipe, in which the staves were deeply rotted, by cutting away the decayed portions, wedging under the bands with pieces of wood, and applying 2:1 Portland cement mortar reinforced with No. 10 iron wire of the sort ordinarily used for tying concrete forms.

Where plain tongues (Fig. 8) are used, water collects in the space between the ends of staves near the top of the pipe. In very cold weather this freezes and eventually splits the upper half of the staves outward, hastening decay and accentuating leakage. Packing the spaces with bituminous material and providing a protective cover of galvanized iron slightly wider than the stave and long enough to slip under and be held by two or more loosened bands, which are then tightened down over them, makes a satisfactory repair. The use of a butt joint similar to Fig. 9 when the pipe is erected will largely, if not entirely, prevent this condition.

B. Concrete Pipe

19. Advantages and Limitations of Concrete Pipe. The advantages of reinforced-concrete pipe are long life and freedom from maintenance.

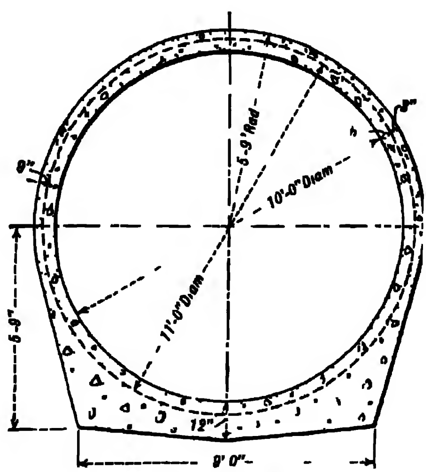
Concrete pipelines which have been in service 25 to 30 years are in as good condition as when new.

Reinforced-concrete pipe is used extensively in connection with waterworks but not so extensively in hydroelectric developments. In the larger sizes usually required in hydroelectric work, wood-stave pipe and steel pipe have generally proved to be less expensive. However, with the development of high-pressure precast reinforced-concrete pipe for heads as high as 600 ft, there are projects in which such pipe would prove economically desirable.

Two possible applications of reinforced-concrete pipe to hydroelectric developments are:

(*a*) *High-line Conduits*, generally under heads of 30 to 60 ft. A 12.5-ft-diameter precast reinforced-concrete pipe has been used under such conditions.

(*b*) *Penstocks*, under heads ranging from 50 ft upward. The use of reinforced-concrete pipe is here probably economically limited to moderate heads of 100 ft or so and to relatively small diameters.



CONSTRUCTION QUANTITIES
Concrete 1 24 cu yds per lin ft
Longitudinal Reinforcement
17-5/8" diam rods 36 - 11 long lapped 3 -2" spaced about 2' 0"
Transverse reinforcement spaced 4 c to c

Head	Rods	Length
70	5/8" diameter	36'-11"
45'	5/8"	36 -5"
30	1/2"	35'-10"
15	3/8"	35'-4"

FIG. 18 Concrete pipe for Siphon No 12, Los Angeles Aqueduct.

20. Cast-in-place Reinforced-concrete Pipe. Reinforced-concrete pipe has been cast in place in diameter sizes up to about 20 ft. A typical cross-section of a cast-in-place pipe is shown in Fig 18.

The use of cast-in-place reinforced-concrete pipe is limited to heads of about 100 ft because of the difficulty in obtaining a dense mix under conditions

usually encountered in high-head developments. However, with a watertight tube of welded steel built into the pipe, it has been used for higher heads.

The concrete for cast-in-place pipe should be poured in cold weather to prevent shrinkage cracks due to temperature changes. Expansion joints are not considered necessary if ample longitudinal reinforcement is provided. Construction joints are either keyed or installed with some types of water stops and the longitudinal reinforcement is made continuous through the joint.

21. Precast Reinforced-concrete Pipe. There is, under many conditions, a material economy in using precast concrete pipe in preference to

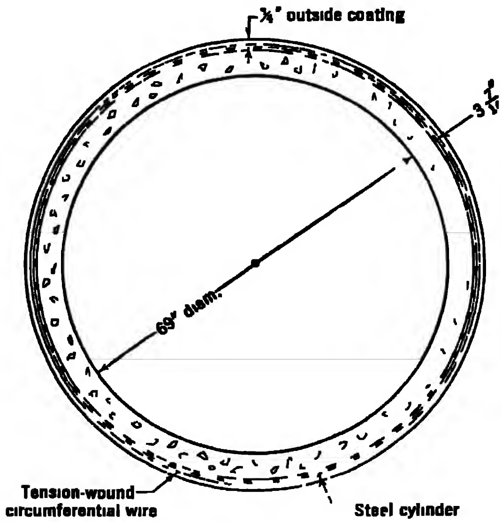
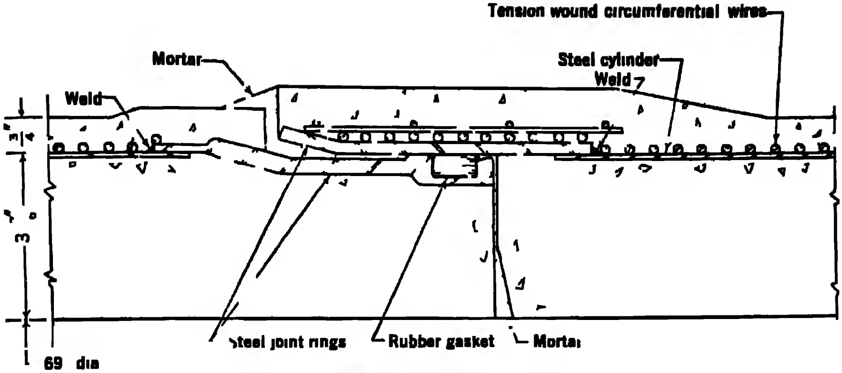


FIG. 19. Cross-section of a 69-in prestressed concrete pipe. (Lock Joint Pipe Co.)

cast-in-place concrete pipe. Precast pipe is made in short lengths, either cast in forms or centrifugally spun, and manufactured locally or at a central plant. A typical cross-section of a precast reinforced-concrete pipe, as made by the Lock Joint Pipe Company, is shown in Fig. 19.

In hydroelectric work, precast concrete pipe is used in diameter sizes from 8 to 12.5 ft, and for heads ranging up to 300 ft. The Lock Joint Pipe Company has developed a concrete pressure pipe, consisting of a welded steel cylinder surrounded on the inside and outside by reinforced concrete, which has been used for diameters up to 12.5 ft and for the higher heads up to 600 ft. The use of precast concrete pipe requires special attention to obtain smooth and watertight joints.

22. Joints in Precast Concrete Pipe. A joint, to be effective, must be watertight and at the same time should be sufficiently flexible to adjust itself to pipe expansion and contraction, or to normal earth settlement. Figure 20 indicates a typical high-pressure rubber and steel joint for precast



Section through joint

Fig 20 High-pressure rubber and steel joint for a 69 in concrete pipe (Lock Joint Pipe Co)

reinforced concrete pipe A rubber gasket fitted into the ring at the spigot end provides a watertight contact when the bell and spigot ends telescope each other The cavity inside and outside the pipe is filled with mortar to make a continuous smooth surface

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CHAPTER 33

TUNNELS

1. General. A general outline of the purpose and use of tunnels in hydroelectric developments and a discussion of those features of tunnels that are common to all types of conduits are presented in Chapter 28.

2. Preliminary Investigation. A prior knowledge of the subsurface conditions on the line of a tunnel will influence the design, construction, and cost of the tunnel. For this reason it is necessary to have a thorough preliminary report before definitely locating the alinement of the tunnel. On any important project such a report is based on (1) topographical surveys, (2) geophysical surveys, (3) core borings, and (4) geological studies. (See also Chapter 7, Investigation of Sites.)

3. Tunnel Geology. Generally a tunnel in firm, hard rock with no leakage would be a most economical structure as far as safety, ease of construction, and maintenance are concerned. However, in tunnel work problems of a geologic nature are sometimes encountered which increase the difficulties and costs of tunnel construction.

All firm hard rock is traversed by fractures appearing as joints, faults, and stratifications. Joints may be a source of leakage, depending upon their tightness. If not broken up badly, jointed rock usually is stable structurally. Faults usually are a source of trouble in tunnel work. Fault zones vary from a few inches to many thousand feet in thickness and are composed of shattered rock and gouge * which may require a lining if pierced by a tunnel. In areas of folded rock, the crest of a fold may consist of fractured rock necessitating support.

The many aspects of the application of geology to tunnel construction are not within the scope of this chapter. Because of the importance of geology, the supervision of subsurface investigations and their interpretation should be left to an experienced geologist.

4. Loading. Methods of determining the internal loading, or maximum internal pressure, for closed conduits are given in Section 3 of Chapter 31. Ordinarily the internal loading in tunnels requires consideration only where the weight of the covering is insufficient to balance the internal pressure. In such cases the tunnel lining must be reinforced.

* Finely pulverized rock, like clay.

5. Determination of Size. For successful operation, the size of the tunnel, for a given discharge, may vary between wide limits; but there is usually one size that will make for the greatest economy of design (Section 4, Chapter 28). However, there is a practical consideration in connection with tunnels. If a tunnel is just large enough so that the most economical equipment may be used for its construction, it is often cheaper per linear foot than a tunnel of a much smaller diameter which is too small to use such equipment. For an unlined tunnel in good hard rock it is probable that a 14- or 15-ft diameter is more economical than any other size under the prevailing high-cost labor conditions in the United States. If the tunnel is lined, the economic equation must be modified, and, in most cases, the cheapest tunnel per linear foot would be of a much smaller diameter. The minimum size tunnel for efficient machine excavation is about 8×8 ft.

The wide range of possible diameter sizes is shown by the fact that the Hoover Dam diversion tunnels are 50 ft in diameter and the 4.5-mi Rove diversion tunnel near Marseilles, France, completed in 1927, is 72 ft wide and 50 ft high.

Practically any velocity consistent with economic design may be used in tunnels.

6. Shape of Section. The shape of the section through absolutely stable material may be whatever proves the most economical from a construction standpoint. When the material is not absolutely stable, the shape of the section must be such that the lining affords the best resistance to the external pressures. This varies with the nature of the materials through which the tunnel is driven, and no standards have been generally adopted. Typical tunnel cross-sections are indicated in Figs. 1 through 4.

The roof of the tunnel is almost invariably supported by a semicircular arch. For hard rock having no tendency to lateral movement, vertical sides may be adopted as in Fig. 1, and the bottom may be horizontal.

Firm-earth or soft-rock tunnels having a slight lateral pressure are provided with a horseshoe-shaped section as in Fig. 2.

Tunnels through very soft earth and tunnels subjected to unbalanced external water pressure must be circular, as in Figs. 3 and 4, or nearly so. It must be remembered that tunnels are frequently emptied and the elevation of the ground water is often much increased by leakage or seepage from the reservoir.

In some instances drains have been provided in the lining to relieve the external pressure when the tunnel is emptied. Such drains, however, are objectionable if they may cause excessive leakage when the tunnel is full.

Tunnels in rock subjected to unbalanced external water pressure need not be circular, but the lining must be figured to withstand such pressure by arching. A horseshoe section with an inverted-arch bottom is in common use for moderate external pressure.

The bottom of an earth tunnel must be designed to support the weight of the tunnel and the top load without settlement. It must therefore be very

thick or shaped as an inverted arch to distribute the side wall loads over the whole base.

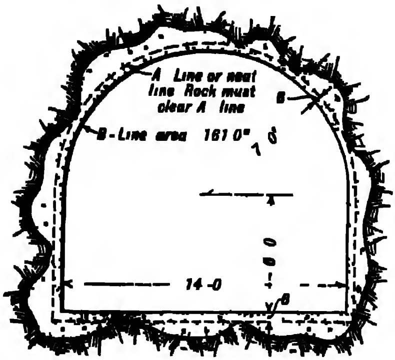
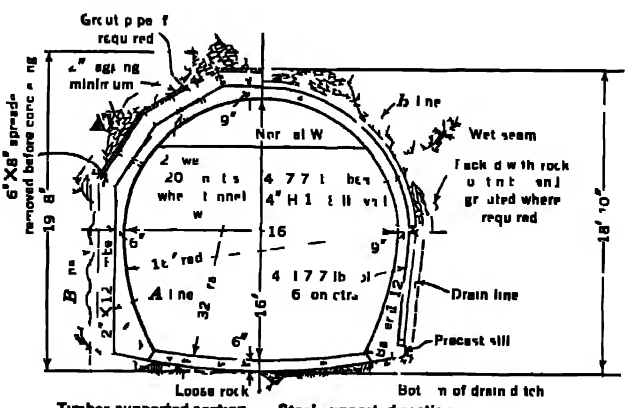


FIG 1 Davis Bridge tunnel New England Power Co

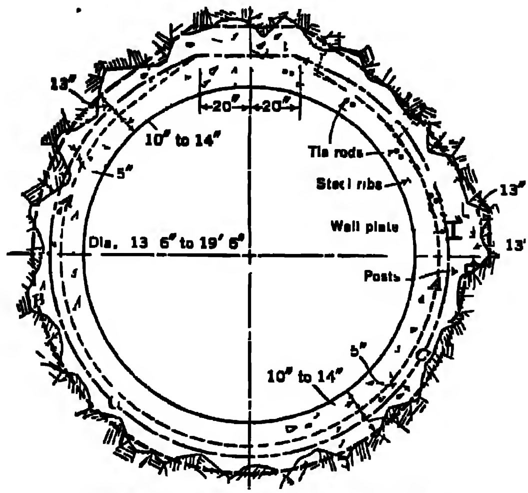
Water cross section area of tunnel as designed	161	sq ft
Average water cross section area	162.5	sq ft
Actual minimum water cross section area	160.5	sq ft
Actual maximum water cross section area	163.7	sq ft
Actual cubic yards of solid excavation per linear foot	7.8[illegible]	sq ft
Minimum thickness of concrete lining permitted	6	in
Average thickness of concrete lining per linear foot	1[illegible]	in
Average cubic yards of concrete lining per linear foot	1.83	



A line is the line within which no portion of support shall remain
B line is the payment line for excavation

FIG 2 Section of the Colorado River Aqueduct tunnel (R M Merriman *Eng News-Rec*, July 26 1934 p 100)

Tunnels reinforced for internal pressure must be of a shape that will permit circular reinforcement, but the outside and inside of the lining may vary from a circle for the sake of economical construction, provided the circular reinforcement is well imbedded



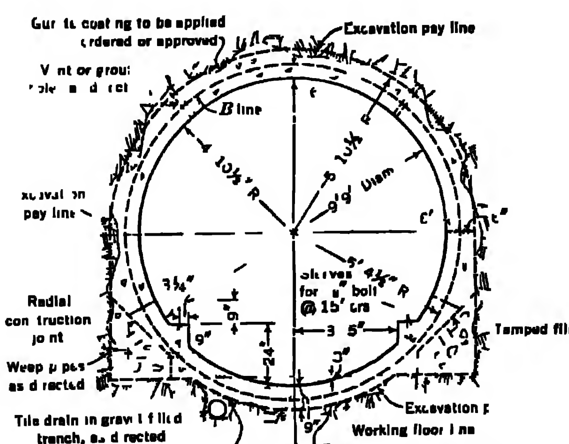
Without roof support | With steel roof support

The *A* line is the line beyond which no rock is permitted to project and contains no permanent support except satisfactorily spaced steel ribs without lagging.

The *B* line is the line for measurement for payment of both excavation and concrete lining

The *C* line is the line of effective thickness of the concrete lining No nonpermanent material is allowed within this line and no large areas of rock, steel, or other imperishable material

Fig 3 Delaware Aqueduct tunnel (R W Armstrong *Jour New Engl Water Works Assoc*, Vol 55 No 2, p 135, June 1941)



Note *B* line is the line within which no unexcavated material or tamped fill shall remain

Fig 4 Continental Divide tunnel (U S Bureau of Reclamation)

7. Depth of Tunnel. In general, the tunnel alinement should be chosen so as to get below faulted, broken, or disintegrated areas, or else to miss them by being located to one side. Most ledge surfaces are likely to be quite badly broken up by weathering cracks near their tops. Consequently, it is sometimes necessary to go some distance below the rock surface in order to secure sound rock.

Unless the tunnel lining is reinforced against bursting pressures, the minimum allowable depth from the ground surface to the top of the tunnel should be enough to counterbalance the maximum hydrostatic pressure within the tunnel, with no dependence placed on the shear strength of the rock. This criterion will be met and generally somewhat exceeded if the following simple rule is adopted: *The minimum permissible depth in feet from the ground surface to the top of the tunnel shall not be less than $\frac{7}{10}$ of the maximum hydrostatic head in feet on the interior of the tunnel, unless the tunnel is reinforced to take the bursting pressure.*

8. Tunnel Support. In unstable ground, supports are erected in the tunnel bore to hold the surrounding material in place until the permanent lining is in place. In tunnels in earth the supports may be of a temporary nature since the lining operation usually follows closely the excavating operation. Tunnels in rock are generally excavated throughout their length before the permanent lining is begun, and the supports must hold the ground until the lining is in, which may take from several months to several years.

Timber and steel are commonly used for the support structure. Such a structure may consist of timber ribs with timber lagging or steel ribs with lagging of either timber or metal. In sand or running ground, steel ribs may be used with steel liner plates or corrugated metal lagging. Steel ribs are usually more expensive than timber, but in long tunnels the saving in amount of excavation and tunnel lining may justify their use. The size and spacing of supports are determined by experience on the job as the tunnel construction progresses.

Both timber and steel deteriorate when left exposed in wet tunnels; consequently, they should not be relied upon for permanent support. However, steel support incased in concrete can be designed as part of the lining and in this way will last indefinitely.

The empty spaces between the lagging and the rock surface must be backpacked and usually grouted in order to provide an even distribution of loads on the finished lining and to prevent the gradual deterioration of the surrounding rock. The type of backpacking depends upon the condition of the rock; it may be timber, gravel, rock spoil, or, especially in pressure tunnels, concrete grout.

Gunite has been used to stabilize and support the rock surface. It is used mainly on lightly fractured rock and usually is the permanent lining of the tunnel.

9. Tunneling in Earth. Tunnels may be driven through almost any material in nature, but the methods used and the costs differ radically. The

method used in tunneling in earth depends chiefly on the bridge-action period of the material above the roof and the position of the water table. Often a water-power tunnel must be started in earth and continued until ledge rock is reached in the hillside; or the tunnel must pierce a formation of soft sediments or crushed decomposed rock. Under such conditions one of the following earth tunneling methods is used (Sections 10, 11, and 12).

10. Shield Method. In this method tunnel driving is done by means of a steel shield which is forced ahead by hydraulic jacks resting on flanges of the cast-iron or steel segments that usually form the lining of the tunnel. The type of shield used depends upon the nature of the material excavated. In self-supporting materials, such as dry clay, cemented gravel, or shale, the open-type shield is generally employed. As this shield advances, the sharp cutting edge breaks down the material into the tunnel where it is removed by conveyor belt, mucking machine, or other means. In materials such as soft clay, silt, or liquid mud, the bulkhead type of shield is used. The bulkhead or driving face is pushed into the material, and the muck is forced into the tunnel through ports.

Subaqueous tunnels through soft ground are generally driven under compressed air. The shield features an air lock which serves as a transition between the pressure side where the excavation is being done and the side open to the atmosphere. The purpose of the compressed air is to support the ground until the lining is in place.

Tunneling by the shield method is very expensive.

11. Liner Plate Method. This method readily adapts itself to tunneling in any raveling material which can stand up unsupported in an opening equal to the area of the liner plate until the plate is bolted in place. Pressed-steel corrugated liner plates 16 in. wide, 36 in. long, and $\frac{1}{8}$- to $\frac{3}{8}$-in. thick with all four edges flanged over 2 in. are used in this method of tunneling.

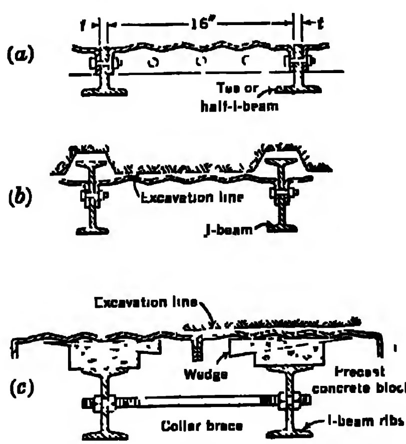


FIG. 5 Assemblies of liner plates and ribs. (From Richardson and Mayo, *Practical Tunnel Driving*.)

For each ring, the installation of liner plates begins at the top center. The top breastboard is removed, and a surface area is trimmed to the size and contour of the liner plate. The liner plate is fitted and bolted to the preceding course. More breastboards are removed and the excavation is widened so that adjacent liner plates can be set on each side. Each time that breastboards are removed they are reset 16 in. ahead following the excavation for the liner ' ' The process is continued until

one course has advanced sufficiently to allow the beginning of the next one, or until the whole ring is completed before beginning on the next one. The liner plates are staggered to break the joints.

In tunnels larger than 10 ft in diameter, it is necessary to stiffen the liner plates with ribs, as indicated in Fig. 5. The T-shaped ribs of Fig. 5(*a*) and the I-shaped ribs of Fig. 5(*b*) are bolted between the flanges of each ring of liner plates. The ribs of Fig. 5(*c*) are braced within the liner plates and can be advantageously incased in concrete as part of the permanent lining.

There are many innovations in the method of erecting liner plates. The liner plate method is faster and sometimes more economical than the forepoling method (Section 12).

12. Forepoling Method. The forepoling method of tunneling, as indicated in Fig. 6, is one method of driving through running ground. Timber

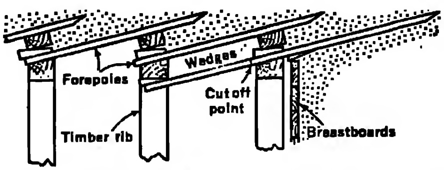


Fig. 6. Forepoling method of tunneling in running ground.

is used exclusively in this type of tunneling, but sometimes steel ribs may be used for supports. Figure 6 shows the method of using poling boards.

Many individual techniques are used in the forepoling method of tunneling. This method is a slow and tedious operation which requires a skilled and experienced gang for safe and satisfactory completion. It is generally used in short stretches of running ground.

13. Tunneling in Rock. Tunneling in firm ledge rock, which does not require support, is generally much cheaper than tunneling in any other material. Methods differ materially with the nature of the rock and with local conditions. The following methods (Sections 14, 15, and 16) are most common.

14. Full-face Method. In this method the full size of the tunnel is blasted out in each round. The development of tunneling equipment, including the drill carriage, has made the method economically possible for all sizes of tunnels. The diversion tunnels at Hoover Dam, the 91 miles of tunnels of the Colorado River Aqueduct, and the 85 miles of the Delaware Aqueduct are notable examples of large tunnels in rock driven by the full-face method.

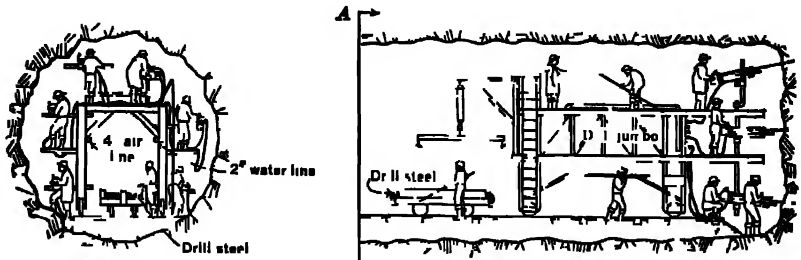
The cycle of operations in the full-face method of excavating in rock, as indicated in Figs. 7 to 14, is as follows:

(*a*) *Drilling.* The drilling, as shown in Fig. 7, is done by automatic-feed drifter drills mounted on a drill carriage or "Jumbo."

The pattern of drill holes determines the amount of ground broken in the subsequent shooting operation and depends upon the size of tunnel and the

type of rock. Figure 8 indicates a typical drilling diagram for the Colorado River Aqueduct 16-ft-diameter tunnels. Numbers refer to the order of firing.

The depth to which holes are drilled varies generally from 5 to 15 ft and is rarely greater than the width of the tunnel. The drill holes indicated in Fig. 8

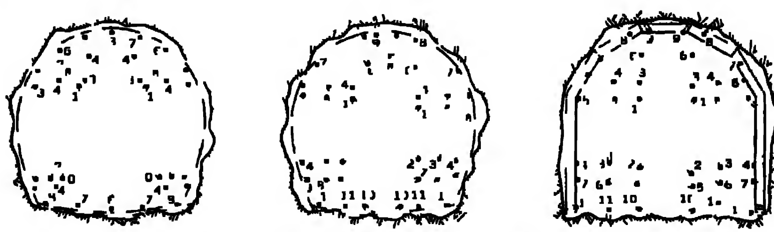


Section A-A   Drilling

A full faced heading is drilled with automatic-feed drifter drills mounted on a drill carriage or "jumbo." This consists of a steel frame about 30 ft long mounted on flanged wheels which operate on steel rails laid about 9 ft apart. The jumbo has an upper and a lower platform and is fitted with movable columns and horizontal arms of heavy steel pipe on which the drills are mounted.

FIG. 7. Drilling a full-faced heading. (R. W. Armstrong, *Jour. New Engl. Water Works Assoc.*, Vol. 55, No. 2, plate II, June 1941.)

ranged from 6 to 10 ft. In the Delaware Aqueduct 19.5-ft-diameter tunnels, the cut holes were 12 to 14 ft deep. The number of holes depends upon the size of the tunnel and the texture and hardness of the rock, as indicated in Fig. 8.



(a) Untimbered section, soft rock 34 hole 6 to 9 ft deep

(b) Untimbered section, hard rock 38 to 50 holes 6 to 10 ft depth

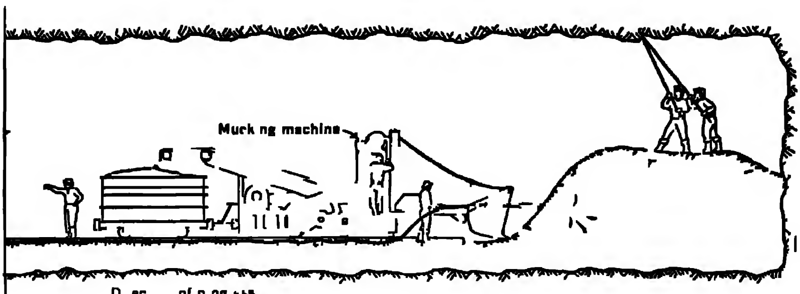
(c) Timbered section 30 to 40 holes 6 to 9 ft deep

FIG. 8. Typical drilling diagram for a full-faced heading Colorado River Aqueduct tunnels. (Numbers denote the order of firing.) (R. M. Merriman, *Eng. News-Rec.*, July 26, 1934, p. 104.)

(b) *Shooting.* In preparation for firing the heading, the jumbo is moved back about 150 ft to avoid damage by flying rock. The amount of explosives used in loading the holes varies with the job; it ranges from about 1 lb per cu yd of material for gravels and soft rock to about 7 lb in hard rock formations. Gelatin dynamite of 40 to 60% strength is generally used for blasting

Firing is done in an order which will break up the maximum amount of ground, as indicated in Fig 8

(c) *Ventilating* Immediately after firing, the fans are reversed from blowing to exhaust for about 15 to 30 minutes to draw off the smoke and dust



After the heading has been shot the Conway mucking machine is moved up to the muck pile and starts loading muck into cars This is accomplished by the machine forcing the digging bucket into the muck and then raising the dipper and chute by hoist chains allowing the muck to slide back onto the conveyor belt where it is carried to the rear and dumped into the car

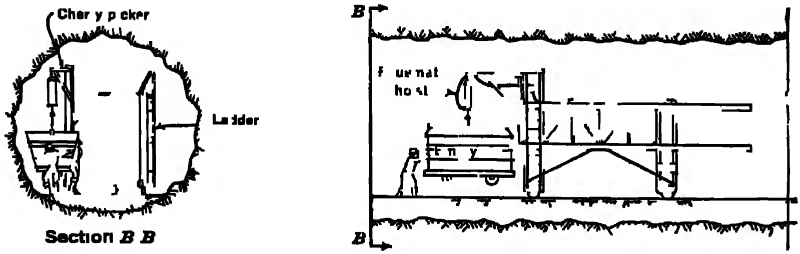
FIG 9 Mucking a full face heading after firing (R W Armstrong, *Jour New Engl Water Works Assn* Vol 55 No 2 plate II June 1941)

(d) *Mucking* As soon as the atmosphere at the heading is cleared the mucking machine is moved up to the muck pile as indicated in Fig 9 Many different mucking machines are available on the market but the Conway

FIG 10 Conway mucking machine (Goodman Manufacturing Co)

machine shown in Fig 10 has been the most popular on the tunnel jobs large and small This type of machine is very efficient in cleaning up fly rock and digging out the corners

The speed of the mucking operation is directly influenced by the car-changing time. In the Delaware Aqueduct, car changing was accomplished by a swinging boom mounted on the drill carriage as indicated in Fig. 11.

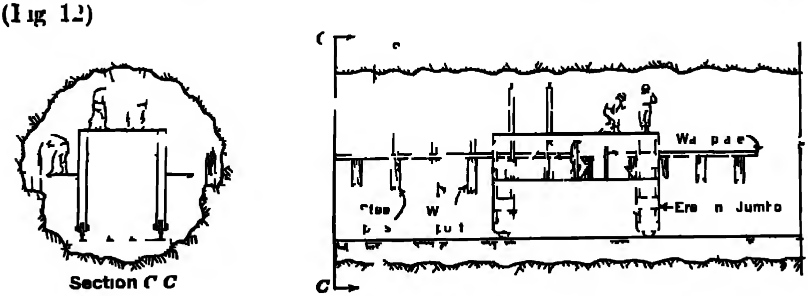


NOTE

During the mucking operation the drill carriage is placed about 150 ft. from the heading. An air jack on a swinging boom mounted on the rear of the drill carriage near the top serves as a "Cherry picker" to lift the last empty muck car and swing it to one side clear of the track until the electric locomotive pulls the loaded cars clear; when the empty car is swung back, lowered to the track and pushed up to the mucking machine to be loaded. This operation is repeated until all cars are loaded. Then they are hauled to the foot of the shaft where one at a time is run into the rotary dumper which rotates about 180 degrees to discharge the muck into a skip. With two car loads the skip is hoisted to the surface and automatically dumps into a hopper from which the muck is chuted into trucks and hauled away for disposal.

Fig. 11. Cherry-picker car changer. (R. W. Armstrong, *Jour. New Engl. Water Works Assoc.*, Vol. 55, No. 2, plate II, June 1941.)

(*e*) *Erecting Support.* If the tunnel requires support, as discussed in Section 8, the support is erected from the drill carriage after each advance (Fig. 12).



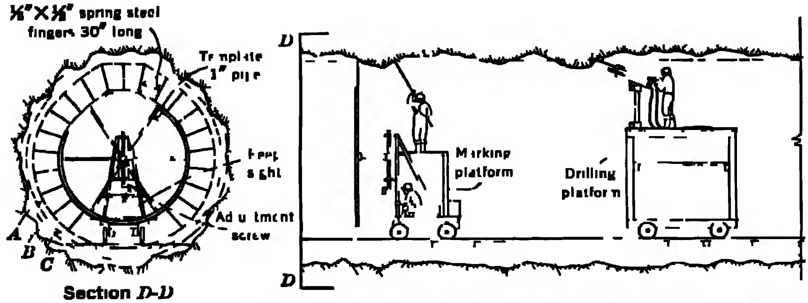
Where rock is poor, steel roof support is erected by the mining crew after each advance from the drill Jumbo and the support is carried well up to the heading. Where the tunnel was supported at time of excavation but later shows signs of raveling due to exposure, steel support is then erected from a special Jumbo.

Fig. 12. Erecting roof support. (R. W. Armstrong, *Jour. New Engl. Water Works Assoc.*, Vol. 55, No. 2, plate II, June 1941.)

(*f*) *Trimming.* As shown in Fig. 13, a special crew trims the excavation to the neat dimensions of the tunnel.

(*g*) *Cleaning Invert.* Some excavated material is allowed to remain in the invert of the tunnel to provide a level working surface for the preceding tunneling operations. After the bore is tunneled through, this material is cleaned

up before the lining is put in place The method shown in Fig 14 was developed on the Delaware Aqueduct It consists of converting the mucker into a slusher by means of a long boom attached to the mucker and using a drag bucket to pull the muck into the mucking machine.



A template made of 1-in. pipe and mounted on a flat car is used for marking "tights" Operator sights through a peep sight at 2 lights, ahead, on ℄ and 3 in below spring line Template is arranged for adjusting so that it may be kept on line and grade.

FIG. 13. Trimming tunnel to neat dimensions (R W Armstrong *Jour New Engl Water Works Assoc*, Vol 55 No 2 plate II June 1941)

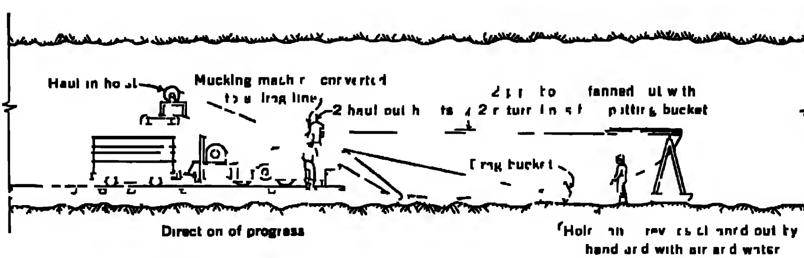


FIG 14 Cleaning the invert of an excavated tunnel (R W Armstrong, *Jour. New Engl Water Works Assoc*, Vol 55 No 2 plate II, June 1941)

15. Heading-and-bench Method. This method is still used on some large tunnels. As indicated in Fig 15, the top heading is kept about the length of one round (5 to 15 ft) ahead of the bench The holes in the top heading are drilled from vertical columns set up on the bench, and those in the bench are drilled with tripod or wagon drills This method has the advantage of providing a bench for erecting support if required; also, both drilling and mucking operations can be carried on at the same time

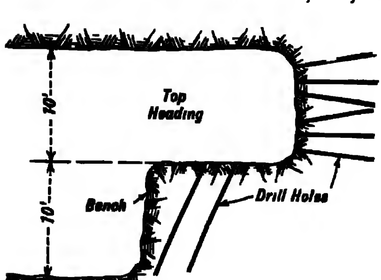


FIG. 15. Heading-and-bench method of tunnel driving

16. Top-heading Method. The top-heading method is used occasion-

ally when a stretch of bad ground needing support is encountered. It is similar to the heading-and-bench method except that it has a longer top heading, usually 50 to 75 ft and sometimes the whole length of tunnel. This provides a good working platform for the support operation

Other variations of the heading-and-bench method are possible but are rarely used.

17. Rate of Progress in Tunneling. If the most economical equipment is used in driving a tunnel, there is a greater rate of progress at a lower cost of excavation. However, the speed at which a tunnel advances depends to a large degree upon the ground conditions encountered. Table 1 gives the rate

TABLE 1

AVERAGE DAILY RATE OF PROGRESS PER HEADING FOR SOME TUNNELS IN ROCK

Tunnel	Location	Date	Total Length, miles	Cross-section of Excavation: Height, feet	Cross-section of Excavation: Width, feet	Material Excavated	Average Advance per Day (Three 8-hr Shifts), feet
Colorado River Aqueduct (dry)	California	1939	72	17.25	17.0	Granite, gneiss sandstone, conglomerate	21.1
Colorado River Aqueduct (wet)	California	1939	20	17.25	17.0	Granite, gneiss (fault zones)	11.1
Carlton	Colorado	1940	6	11.0	11.0	Granite	47.2
Delaware Aqueduct	New York State	1941	85	17.0 to 19.0 diameter		Gray sandstone	31.62
Continental Divide	Colorado	1942	13	11.75 to 12.75 diameter		Granite, schists	52.0
Appalachia	Tennessee	1942	8	18.0 to 22.0 diameter		Quartzite	27.7

of progress for a 24-hour day per heading for a few typical tunnels driven in rock. A definite comparison cannot be made because tunneling conditions vary widely. The maximum advance for the tunnels given in the table ranged from 53 to 65 ft per day.

18. Overbreak and "Pay-line." *Rock Tunnels.* In preparing a preliminary estimate of the cost of excavation and concrete lining that will be required for a tunnel in rock, it is necessary to include a proper allowance for overbreak. As here used, overbreak means the rock that is actually excavated in addition to that which is included within the required neat line of excavation of the tunnel. Thus, as in Fig. 16, the cross-sectional area of a concrete-lined tunnel, that is, the water-carrying area, might be 154 sq ft, while the area to the line beyond which no rock is allowed to project, called the neat line of excavation, might be 200 sq ft, and the area inclosed by the line to which the excavation actually breaks is, say, 250 sq ft. The percentage of overbreak is, then, 50/200, or 25%. The percentage of overbreak is sometimes, but incorrectly, stated as the percentage of overbreak over the tunnel section, or water-carrying area.

The actual amount of overbreak depends very largely on the nature of the rock and the skill with which the drilling and shooting are done. For rock tunnels requiring support that cannot be removed before the lining is placed, the overbreak is greater by the space outside the neat line to be provided for the support.

In unit-price contract work, it is necessary to limit the overbreak, because with excessive overbreak the rock can be removed and the thicker lining placed at less cost per cubic yard. Therefore, it is customary in writing unit-price contracts to provide outside limits to the excavation and concrete, called the "pay-lines," beyond which no excavation or concrete will be paid for.

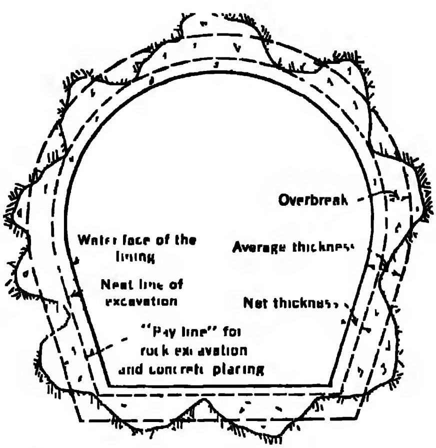


FIG. 16. Overbreak and pay-line for a tunnel.

For unsupported tunnels, the pay-lines for excavation and concrete are usually identical. They are usually placed a constant distance outside the neat line of excavation and are fixed as near as possible to the probable line of average actual excavation (Fig. 16) which would be consistent with reasonable care.

Table 2 shows the average overbreak d for a number of tunnels. It will be noted that, except for a few cases, the overbreak averages about 12 in.

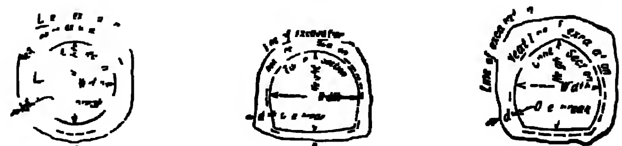
For rock tunnels having supports left in place, the pay-line for excavation at the sides and top is moved out a distance equivalent to the space estimated to be required for the supports. This space varies greatly according to the size of the tunnel and the nature of the rock.

The pay-line for concrete in supported rock tunnels is inside the pay-line for excavation by a distance corresponding to the estimated volume of the supports required.

Overbreak is far from constant and should be based on the nature of the rock. A firm igneous or metamorphic rock with few seams can usually be made to break closer to the neat lines than a sedimentary rock; and a sedi-

TABLE 2

Studies in Average Overbreak in Excavation for Tunnels



A B C

No	Tunnel	Shape of Section	Dimensions: Tunnel Section Ht	Dimensions: Tunnel Section Width	Dimensions: Neat Excavation Ht	Dimensions: Neat Excavation Width	Areas—Sq Ft: Tunnel Section	Areas—Sq Ft: Neat Section	Areas—Sq Ft: Actual Excavation	Percentage Overbreak over Neat Section	Overbreak d feet	Nature of Material Excavated	Type of Support if Supported
1	Weston Aqueduct	B	9 3	[illegible] 0"	10 0	10 9"	76	88	127	44 4	1 07	Coarse granite	Partly timber
2	Wachusett Aqueduct	B	16 10	[illegible]	1[illegible]	13 6"	108	13[illegible]	172	31 6	0 38	Shaly schist	
3	New Croton Aqueduct	A	10 6"	10 6	13 2"	13 [illegible]	88	146	222	50 4	1 48	Limestone	
4	New Croton Aqueduct	B	13 $7\frac{1}{4}$	13 6	15 6	15 $7\frac{1}{4}$	154	203	[illegible]4	35 0	1 25	Gneiss	
5	South Metropolitan High Level Sewer	C	9 [illegible]	8 3	10 6"	9 7	59	7[illegible]	116	47 5	1 02	Conglomerate	
6	Davis Bridge Plant	B	13 0	14 0	14 0	15 0	151	186	[illegible]12	14 0	0 50	Mica schist	
7	Nantahala	B	14 0	13 0	14 0"	13 0	164	164	[illegible]04	24 0	0 [illegible]8	Hard gray wackes	
8	Indian Rock Dam	A	15 0	15 0	15 0	18 0"	177	254	347	36 5	1 50	Quartzite phyllite	
9	Knightville Dam	A	16 0	16 0	18 0"	18 0	201	[illegible]4	[illegible]14	[illegible]3 6	1 0	Quartzite and micaceous schist	
10	Surry Mountain Dam	B	11 0	10 0	1[illegible] 0	12 0	83	119	1[illegible]	67 0	0 70	Blocky granite	
11	Apalachia	A	18 0	18 0	20 0	20 0	[illegible]54	314	[illegible]	25 0	1 10	Quartzite	Very little timber and steel
12	Main Colorado River Aqueduct	B	16 0	16 0	1[illegible] 3	1[illegible] 0	216	250	304	21 6	0 98	Granite gneiss	Timber and steel (in part)
13	Hoover Dam	A	50 0	50 0	56 0	56 0"	1964	2463	2615	6 1	0 88	Andesite breccia	
14	Rondout West Branch	A	13 6"	13 6"	13 6	15 2	143	194	281	17 0		Shale and sandstones 64%, gneisses and schists 28%, limestone 8%	Unsupported 18%, gunite 39%, steel roof support 43%
15	West Branch Kensico	A	15 0	15 0	15 0"	16 0"	177	221	311	10 0		Gneisses and schists 96%, granite 1%, limestone 3%	Unsupported 2%, gunite 57%, steel roof support 41%
16	Kensico Hill View	A	19 6	19 6"	19 6"	21 6	299	366	512	15 0		Gneisses and schists 41%, granite 51%, limestone 8%	Unsupported 1%, gunite 8%, steel roof support 91%

Average overbreak omitting Nos 3 8 14 15 and 16 — 0 982

mentary rock in which the strata are vertical will, other things being equal, usually break closer to the neat line than sedimentary rock that has horizontal stratification.

Earth Tunnels. The term "overbreak" is not used in connection with earth tunnels, as in such work it is possible to excavate very closely to prescribed lines. The pay-line for excavation in an earth tunnel is at a distance from the neat line equal to that estimated to be required for the supports. This distance varies upward from 4 in., depending upon the size of the tunnel and type of support.

The pay-line for concrete is inside the pay-line for excavation by a distance corresponding to the estimated volume of supports.

19. Tunnel Lining. It is the usual practice to line tunnels with concrete. The purpose of the lining is threefold:

1. To obtain a smooth surface which will give a minimum loss of head.
2. To increase the structural safety of the tunnel.
3. To prevent loss of water by seepage through seams in the rock.

The thickness of lining required depends upon the nature of the material to be supported and the method of construction. Since it is impossible to excavate exactly to prescribed lines, the "line of average excavation" is the average line to which the material is actually excavated; this line also fixes the "average thickness" of concrete lining. The "net thickness" of lining should have a minimum of 4 to 8 in., depending upon the size of tunnel. The "overbreak," or excess material removed beyond the prescribed neat line, for rock tunnels has a minimum of about 8 in. and may be several feet if the tunnel is heavily timbered. Examples of overbreak are given in Table 2. Overbreak averages about 12 in. for stable rock tunnels. Thus the thickness of rock-tunnel lining is required by practical considerations to be a minimum of 12 in. and an average of 16 to 20 in. Therefore it is seen that, unless the tunnel is very large or the rock quite unstable, the minimum thickness of lining which it is practicable to build is of ample strength.

Only under exceptional conditions can earth tunnels be excavated without supports. Under such conditions, however, the amount of overbreak would be very small. Supports frequently are not removed, and, as they cannot be counted on to contribute to the strength of the lining unless they are steel encased in concrete, their area must be deducted from the average thickness of lining.

It is impossible to estimate exactly the external loads to which the tunnel lining will be subjected. Therefore, the engineer must be guided by what experience has proved to be adequate under similar conditions.

For stable rock, a net thickness of linings of 6 in. for small areas to 12 in. for large tunnels has been used. For unstable rock, a net thickness of 8 to 12 in. is common; this, together with 10 to 18 in. of overbreak, results in an average thickness of 18 to 30 in.

Ordinarily, a concrete proportioned for an ultimate compressive strength of 3000 lb will answer the purpose of tunnel lining. When a high degree of

watertightness is required, richer mixtures giving higher ultimate strengths are sometimes used. As the lining is usually thin, a surplus of mortar is frequently used to insure plasticity and the filling of all irregularities. When the minimum thickness of lining is 6 in., the maximum size of aggregate should be limited to 1 in. When the minimum thickness of lining is 12 in., the maximum size of aggregate should be limited to 1½ in.

Sometimes steel pipe is incased in the concrete lining in place of reinforcing steel.

The usual procedure in lining rock tunnels is to place the invert first and thus provide a firm base from which to aline the side and arch forms. The only forms required for concreting the invert are those forming the longitudinal joint at the sides. They also serve as guides for the screeding and finishing operations. The sides and arch of a tunnel lining are poured in one operation, either as a continuous lining or in sections. Many long tunnels have been poured preferably with a continuous side and arch lining because of the freedom of construction joints. This was made possible by the improvement in telescoping forms which permit an easy dismantling and resetting operation. Concreting is generally done by some form of the "concrete gun" or Pumpcrete methods. Ordinarily these methods are the most economical and desirable for tunnel work.

20. Tunnel Grouting. Grouting in rock tunnels before the concrete lining is in place is done either to consolidate the surrounding rock for support (Section 8) or to seal off water (Section 22). Grouting after the lining is in place is done to fill the voids between the concrete lining and the rock to prevent load concentrations on the finished lining. Holes left through concrete lining for grouting should be large enough to permit use of jackhammers, and drilling should be done at least 1 ft into rock before grouting. In the Delaware Aqueduct tunnels,* grouting back of the lining was done under pressures up to 600 lb per sq in. for the purpose of filling all voids; filling and consolidating; backpacking; plugging drains and weepers; and completely filling joints, cracks, fissures, and other voids in surrounding rock.

The grouting procedure depends upon the geological features of the locality and is similar to that described in Section 3c of Chapter 25.

21. Shafts and Adits. If the proposed tunnel is long and the progress schedule severe, the cost of hauling the muck long distances underground usually requires that the tunnel be broken up into a number of different sections and worked simultaneously from different headings. To reach these headings several construction shafts are sometimes required. Sometimes the tunnel can be reached by means of adits or drifts in the hillside, which are merely side tunnels driven from the face of the hill to the alinement of the tunnel to permit starting additional headings. In locating the alinement of a tunnel, the ease of access for construction purposes should be given due consideration. Other things being equal, it is cheaper to locate

* See Ref. 10 of Section 24.

the tunnel so that it can be reached at several points in the line by means of drifts at grade, instead of placing it back in the hill where construction shafts of great depth might be required.

Shafts are usually either circular or rectangular in section. If a considerable depth of soft ground must be penetrated before ledge rock is reached, the shaft is frequently sunk as a steel or reinforced-concrete open caisson. In extreme cases, pneumatic caissons are used. Sections of the steel or reinforced-concrete shaft lining are built up aboveground and sunk by excavating below the bottom or cutting edge of the caisson. When soft ground conditions are not so severe, the shaft may be sunk by the forepoling or liner plate method (Sections 11 and 12); where the material is fairly firm, the earth is simply excavated and the shaft lined with lagging and braced as the work progresses.

When ledge rock is reached, the method of procedure is quite similar to that used in driving a heading. After a set of drill holes have been completed, a "V" is first shot out of the center of the shaft. This provides a space for the other holes to break to when they are shot. Each time, before shooting, everything must usually be lifted out of the shaft. This generally makes shaft sinking a slower process than driving heading. For this reason, shaft sinking ordinarily costs from 1.5 to 2 times as much per cubic yard of rock taken out as tunnel excavation in similar material and of the same cross-section.

22. Seepage. In the construction of almost all tunnels there is usually seepage water which must be taken care of by one of two methods. In one method, grouting under pressure (Section 20) is used to seal off the water-bearing strata. The other method consists of a system of small drifts or collecting pipes placed close to the sides of the tunnel to catch the seepage. This drainage system may be effectively used to drain the surrounding material so that the tunnel can be advanced in reasonably dry ground.

23. Ventilation. Proper ventilation, particularly in driving long tunnels, is required to replace underground gases, blasting fumes, and drilling dust with a constant supply of fresh air. The ventilating plant is usually designed to blow fresh air from the outside through light steel or fabric pipe, varying from 12 to 36 in. in diameter. Ventilation is most effective when the intake pipe is brought to within 100 ft of the heading. The pipe is generally supported along the side or ceiling so that it is out of the way, but just before firing a heading the last 500 ft of pipe is made to rest on the floor Sometimes wet drilling in combination with adequate ventilation is necessary for satisfactory control of dust.

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CHAPTER 34

WATER HAMMER

By E. B. Strowger *

1. Definition. Water hammer is defined as the change in pressure, above or below normal pressure, caused by sudden changes in the rate of water flow. Because of sudden changes in the demand for water during load fluctuations, water hammer occurs at all points in penstocks between the forebay or surge tank and the turbines, and to some extent in the conduit between the reservoir and the surge tank.

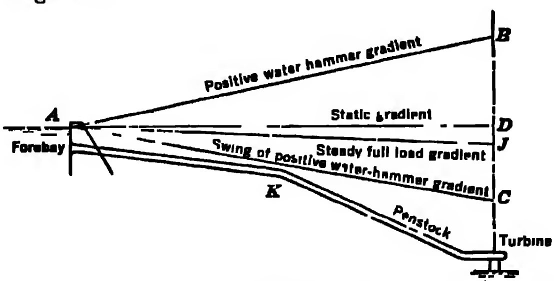


Fig 1. Change in hydraulic gradient with decrease in load.

Accompanying a rather sudden decrease in load on a hydroelectric plant, the turbine gates operate to close and the hydraulic gradient moves up from AJ to AB as shown on Fig. 1. This position is called the positive water-hammer gradient. At the instant the turbine-gate movement ceases, the supernormal pressure then existing becomes unstable and the gradient AB begins to swing to AC. The pressure then fluctuates between positive and negative until damped out by friction

Accompanying a rather sudden increase in load on a hydroelectric plant, the turbine gates operate to open and the hydraulic gradient moves down from EJ to EG, as shown on Fig. 2. This position is called the negative water-hammer gradient. After the gate movement ceases, the negative pressure EG swings to positive pressure EF.

The penstock should be designed to withstand, at every point, an internal pressure corresponding to the maximum positive water-hammer pressure,

* Consulting engineer, Buffalo, N. Y.

AB. The negative water-hammer gradient, *AC* or *EG*, whether caused directly by gate opening or by the swing of pressure from positive to negative after gate closure, should not be below the top of the penstock at any point, as at *K*. If it is, a partial vacuum will occur within the pipe, with possibility of collapse of the shell. Air-inlet valves to prevent vacuum in this case are not recommended, as they may not operate quickly enough to prevent collapse.

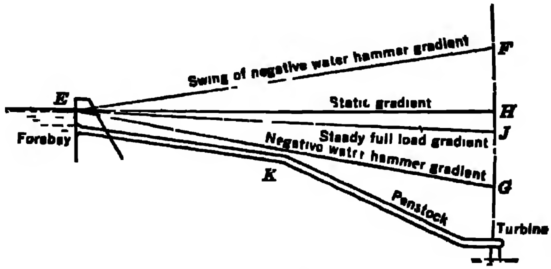


FIG. 2. Change in hydraulic gradient with increase in load.

2. General Discussion. If the turbine gates of the installation shown in Fig. 1 are at a fixed position, as when the governor is operating on load limit, the rate of flow through the turbine orifices is constant and the hydraulic gradient remains in a fixed position. If, however, the turbine gates are regulating, i.e., causing the orifice area alternately to open and close, the column of water is accelerated and decelerated and the hydraulic gradient changes in position. During this perturbed regimen of flow, the phenomenon of water hammer occurs. During a closure or an opening of the gates, kinetic energy is converted to potential energy, or vice versa, and the pressure existing at any instant could readily be computed from Newton's laws of motion if it were not for the effect of elasticity of the penstock walls and of the water itself. As the pressure changes occur, the water is compressed or relieved of pressure, the penstock is expanded or contracted, and water-hammer waves travel along the penstock as the whole liquid column vibrates. Velocity and pressure at any instant thus are dependent on the elasticity characteristics of penstock and water, as well as on the initial conditions of head and velocity, the length of penstock, the velocity change, and the nature of the gate motion.

3. Classification. The American Society of Mechanical Engineers classification of water-hammer problems in connection with hydroelectric plants is

Case 1. Conduits of uniform thickness and diameter (simple conduits)
Case 2. Conduits of variable thickness and diameter (complex conduits).
Case 3. Conduits with branches (compound conduits).

This chapter will be confined to the solution of water-hammer problems involving simple conduits and also complex conduits converted to simple

conduits by approximate methods. For more exact solution of problems involving complex conduits and compound pipes, see graphical methods by Schnyder [4], Bergeron [5], and Angus [6, 10].

4. Simple Conduits. The first complete analytical solution to the water-hammer problem was given by Lorenzo Allievi [2] in 1902. Allievi presented simple charts for the determination of the maximum pressure rise for uniform closure in simple conduits. In 1919, N. R. Gibson developed a detailed theory of water hammer, independent of the work of Allievi but with identical results. Gibson developed his equations on the basis of a series of instantaneous water-hammer waves. His work included two methods of computation, one using his formal equations and the other employing simple arithmetic integration [3].

As the turbine gates start closing, a positive pressure wave starts to travel up the penstock to the forebay. A given motion of the gates can be considered to consist of a great many small motions, each one of which is instantaneous. The positive wave resulting from one of these small instantaneous motions of the gates in the closing direction travels up the penstock to the forebay, and, upon reaching the forebay, it is reflected from the open end of the penstock as a negative wave that travels back to the turbine and that has the same magnitude as the positive wave. The time of one round trip of the wave is designated as "μ" and is called *time of one interval* or the *critical time* of the pipe. It is expressed (in seconds) as

$$\mu = \frac{2L}{a} \qquad [1]$$

where L is the length of the penstock in feet and a is the velocity of the pressure wave in feet per second.

Joukovsky [1] proved that the water hammer produced by an instantaneous change in velocity in a pipe is equal to

$$h = \frac{a\Delta v}{g} \qquad [2]$$

where h = pressure rise, in feet;
Δv = velocity change, in feet per second; and
g = acceleration due to gravity, in feet per second per second

This pressure rise is produced when a velocity of Δv feet per second is destroyed instantaneously or in a time less than the critical time of the pipe. With instantaneous closure Δv would, of course, be measured at the point where the flow is stopped. Equation 2 is the fundamental equation upon which all water-hammer studies are based.

The formula developed for the velocity of the pressure wave is given by

$$a = \frac{12}{\sqrt{(w/g)\,[(1/k) + (d/Ee)]}} = \frac{4675}{\sqrt{1 + (kd/Ee)}} \qquad [3]$$

where d = diameter of pipe, in inches,
e = thickness of walls of pipe, in inches,
k = the voluminal modulus of elasticity of water in compression, 294,000 lb per sq in ,
E = modulus of elasticity of the material of the pipe walls, approximately 29,400,000 for steel, pounds per square inch,
w = weight of 1 cu ft of water, in pounds

For steel pipes commonly encountered in practice, this formula becomes

$$a = \frac{4675}{\sqrt{1 + (d/100e)}} \qquad [4]$$

Figure 3 shows values of a for various values of d/e for steel pipe in accordance with Eq 4 and also for cast-iron pipe For a pipe concreted in solid rock the fraction kd/Ee in Eq 3 becomes infinitesimal and the limiting value of 4675 is reached for a, this being the velocity of sound in water

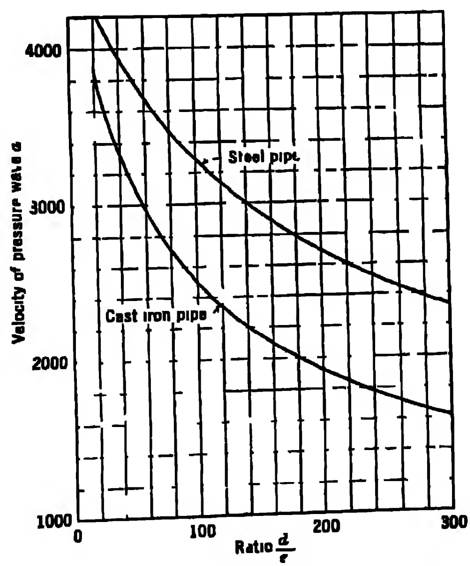


FIG 3 Values of a for various values of d/e steel pipe and cast iron pipe

The value of a for wood-stave pipes depends not only on the modulus of elasticity of the wood making up the staves and that of the steel of the bands but also on the deflection of the staves between bands, the degree the bands have been imbedded in the staves, and perhaps other factors

In general, however, the value of a for wood-stave pipes is low, how low it is impossible to foretell This lack of knowledge would make the problem of water hammer in such pipe indeterminate if, as explained in Section 7,

it were not for the fact that often it makes little difference what value of a is selected. It is suggested that the following formula be used to determine a tentative value of a, that several lower values be taken to investigate what effect a change in a has on the result, and that the highest value of water hammer, so derived, be adopted.

$$a_w = \frac{4675}{\sqrt{1 + [kd/(E_w b + E_s \phi)]}} \tag{5}$$

a_w = velocity of pressure wave in wood-stave pipe, in feet per second.
k = voluminal modulus of elasticity of water in compression
= 294,000 lb per sq in.
d = diameter of pipe, in inches.
E_w = modulus of elasticity of wood staves. For the usual kinds of wood used in wood-stave conduits, the moduli of elasticity are as follows:

B.C. fir or Douglas fir	1,600,000 lb per sq in.
Redwood or cypress	1,330,000 lb per sq in.
White pine	700,000 lb per sq in.

For old staves deteriorated by use and the action of the elements, these values should be reduced considerably; for instance, tests on B.C. fir have shown that the value of 1,600,000 lb per sq in. may be reduced to as low as 420,000 lb per sq in.
b = stave thickness, in inches.
E_s = modulus of elasticity of steel bands in tension
= 29,400,000 lb per sq in.
ϕ = total cross-sectional area of steel bands, in square inch per lineal inch of pipe.

For concrete pipe and buried pipe the value of a usually runs high with a possible upper limit of 4675 ft per second.

5. Complex Conduits. For complex conduits there is a reflection of the pressure waves at points where changes in pipe thickness or diameter occur. A rigid solution of this case would require a consideration of the reflected waves, as is given by means of the graphical method. However, the maximum pressure rise usually occurs near the end of the gate movement for relatively slow closures. If the closure in the case of complex penstocks of hydroelectric plants occurs in a time greater than, say, 5 intervals, and if the gate motion is approximately uniform, the Allievi charts (see Section 7) can, in general, be considered accurate enough for preliminary investigations. As a rule, however, the graphical method should be used for final calculations. With experience one will be able to tell when the Allievi charts will be accurate enough for the final design. To apply the Allievi charts the complex penstock must be reduced to an equivalent simple penstock of uniform thickness and diameter. In this case a value of a should be determined for

each section of constant diameter and thickness, and the effective value of a should be obtained from the following equation:

$$\frac{L}{a} = \frac{l_1}{a_1} + \frac{l_2}{a_2} + \frac{l_3}{a_3} + \text{etc.} \qquad [6]$$

where L = the total length of penstock, including the surge-tank riser; and l_1, l_2, l_3, etc. = successive lengths of penstock having constant values of a_1, a_2, a_3, etc., respectively.

The effective initial velocity, V_0, should be calculated as follows:

$$V_0 = \frac{l_1v_1 + l_2v_2 + \text{etc.}}{L} \qquad [7]$$

where l_1, l_2, l_3, etc. = successive lengths of penstock having constant velocity values of v_1, v_2, v_3, etc., respectively.

In those low-head hydroelectric developments in which the length of draft tube is a relatively large portion of the penstock length, the draft tube should be included with the penstock in the computation of effective velocity.

6. Compound Conduits. The reflection of waves from branch pipes must be considered in almost all cases in order to determine the true maximum pressure rise at any point in the hydraulic system for either rapid or slow closures.

This statement does not apply to the usual case where the penstock branches to turbines near the powerhouse. In hydroelectric practice it applies only where a branch pipe may be dead-ended at the time that closure of all turbines occurs, for example, at the by-pass conduit at Hoover Dam.

In general, the graphical method is best adapted to the solution of such special problems [9 and 10].

7. Allievi's Water-hammer Charts. Allievi's chart for the solution of maximum pressure of water hammer when the velocity is destroyed by uniform gate motion to zero is reproduced in Fig. 4. Figure 5 has been prepared to show the detail of the chart at low values of ρ and θ. Allievi's chart is based on two approximations:

1. It is based on the assumption of uniform wall thickness of the pipe and uniform diameter, in order to eliminate the consideration of wave reflections from points of change in thickness and change in diameter, and thus applies to simple conduits as described above, or to an equivalent simple conduit having a constant value of a and a uniform initial velocity V_0 as explained in Section 5.

2. It is based also on certain gate-closure characteristics explained later.

Allievi's chart for the minimum pressure due to an increase in velocity when the turbine gates open is shown in Fig. 6. The foregoing two approximations apply also to this case. Figure 6 is made for opening the turbine gates from a closed position to any desired degree of opening and at any speed of operation.

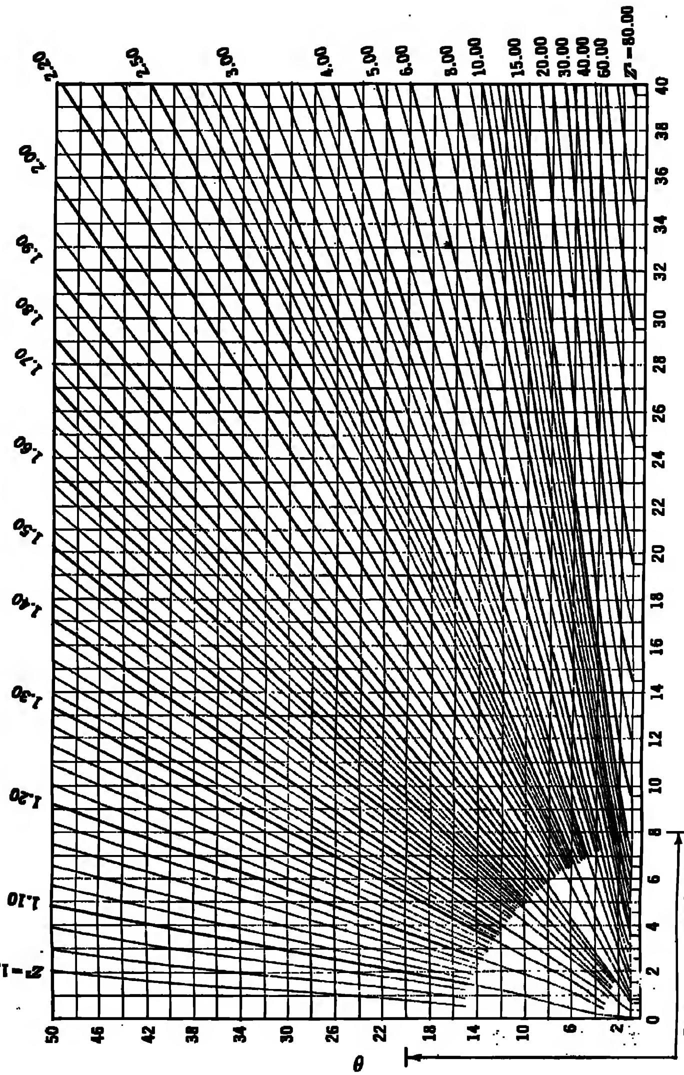


FIG. 4. Chart for the determination of maximum pressure rise, for uniform gate motion and simple conduits (for large values of ρ and θ).

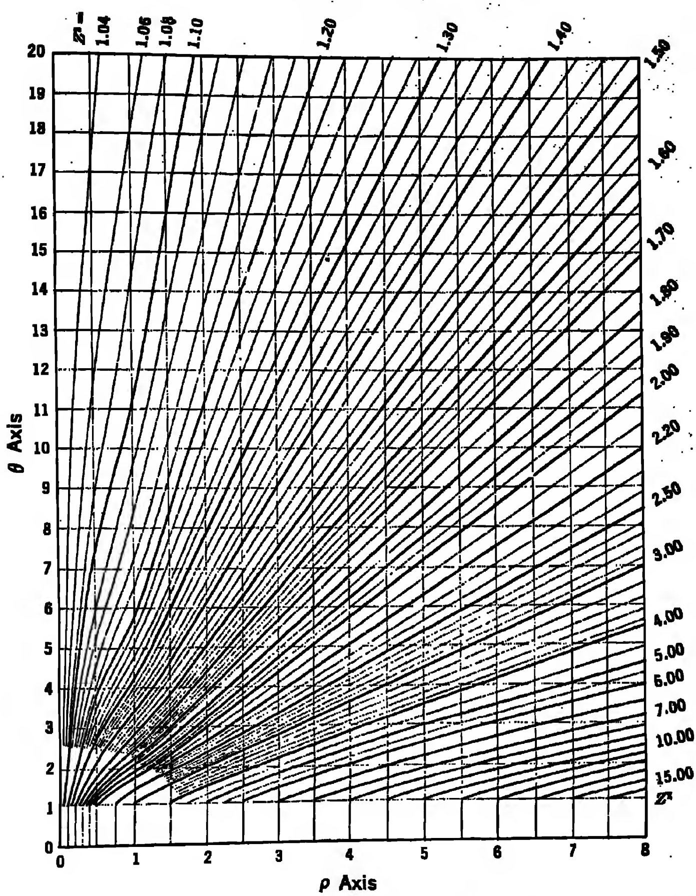


FIG. 5. Chart for the determination of maximum pressure rise, for uniform gate motion and simple conduits (for small values of ρ and θ).

The coordinates of these three charts are dimensionless numbers or parameters and are designated as ρ and θ. The parameter ρ is called the *characteristic* of the conduit and is expressed by the equation

$$\rho = \frac{aV_0}{2gH_0} \tag{8}$$

where H_0 is the static head at the lower end of the penstock, in feet of water, measured from the forebay level or from the elevation of water in the surge tank at the beginning of gate movement to the tailrace level. Other symbols are as previously given.

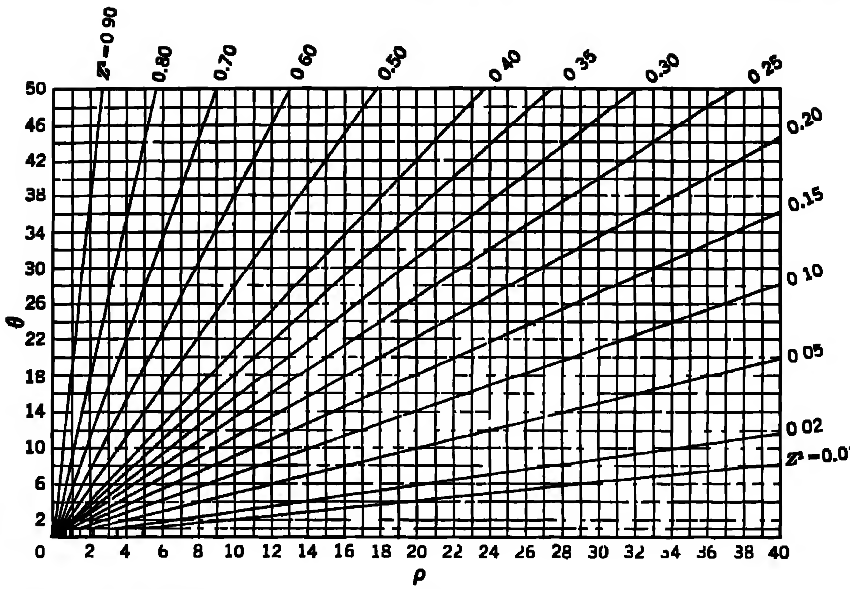


FIG 6 Chart for the determination of maximum fall in pressure for uniform gate motion and simple conduits

The parameter θ, called the *time parameter*, is the ratio of the total time of gate operation to the time of one interval as given by Eq 1, i e, it is the time of gate operation expressed in intervals and can be written

$$\theta = \frac{aT}{2L} = \frac{T}{\mu} \qquad [9]$$

where T is the net equivalent time of gate operation

The symbol Z is a measure of the water hammer Z^2 is defined as the ratio of the maximum total head to the initial head; i e,

$$Z^2 = \frac{H_0 + h}{H_0} \quad \text{or} \quad h = H_0(Z^2 - 1) \qquad [10]$$

where h is the water-hammer head

It should be noted that for a surge tank the calculated water hammer, h, should be increased by the sudden change in water level, h_R, in the riser pipe of a differential tank which occurs during the period of gate movement. Therefore, from Eq 10, the total increase in pressure is

$$h_T = h + h_R \qquad [11]$$

The water-hammer pressure in the system should not be confused with the more or less gradually changing pressure in the surge tank that is due to the mass oscillation of the water and takes place over a much longer period than

the time of gate stroke. This pressure rise in the surge tank may be larger or smaller than that due to water hammer and should also be investigated to see which pressure is controlling in the proper design of the penstock.

Where the closing time is relatively long, say more than 5 intervals, and the gate motion is approximately uniform, the Allievi charts can be used for approximate results in problems involving complex conduits by reducing the complex conduit to an equivalent simple conduit by means of Eqs. 6

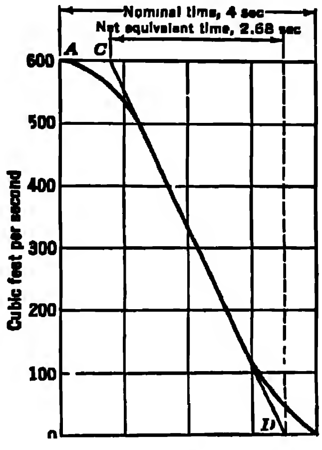


FIG. 7. Typical discharge-time curve for turbine with no penstock (no water hammer).

It should be noted that a appears in the expression for each coordinate of the Allievi chart and that θ and ρ each vary directly with a. Since for all values of θ greater than about 5 the lines on the chart for equal pressure rise (the Z^2 lines) are approximately straight and when extended very nearly pass through the zero of coordinates, it is evident that, for problems involving more than about 5 intervals, a considerable error can be made in the value of a without affecting the result appreciably.

Figure 7 shows a curve indicating a typical basic relation between the turbine discharge and time during the closure of the turbine gates under the condition of zero length of penstock (no water hammer). Allievi's charts assume that this basic curve is a straight line from A to B. The effect of changes in discharge due to water hammer in the penstock is taken care of in his charts.

The straight line CD, tangent to the steepest portion of the curve AB, defines the *net equivalent governor time*, as indicated, for use in Eq. 9 applying to the Allievi charts. The net equivalent time can be obtained from the turbine manufacturer. When it is not available, a conservative value for

preliminary studies is 60% of the nominal governor time. The results thus obtained are approximate; if an exact solution is desired, the arithmetic integration method or the graphical method as described in Sections 9 to 12 should be employed.

8. Example by Allievi Chart. Assume the following conditions:

Section of penstock	1	2	3	4
Diameter, d, ft	9	9	8.5	8
Length of section, ft	100	200	300	400
Thickness, e, in.	$\frac{1}{2}$	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{3}{4}$
d/e	216	144	136	128
a from Fig. 3	2625	2990	3050	3100
Area, sq ft	63.6	63.6	56.7	50.3
Velocity for $Q = 600$ cu ft per second	9.44	9.44	10.58	11.93

$$H_0 = \text{initial head} = 200 \text{ ft}$$

$$g = 32.2$$

$$T_n = \text{nominal governor time} = 4 \text{ sec}$$

(*a*) *Pressure Rise.* Calculate the maximum pressure rise at the turbine for a gate closure to zero as shown in Fig. 7.

From Fig. 7 the net equivalent time is T is 2.68 sec.

From Eq. 7,

$$V_0 = \frac{300 \times 9.44 + 300 \times 10.58 + 400 \times 11.93}{1000} = 10.78 \text{ ft per second}$$

From Eq. 6,

$$\frac{1000}{a} = \frac{100}{2625} + \frac{200}{2990} + \frac{300}{3050} + \frac{400}{3100}$$

Average $a = 3005$ ft per second.

From Eq. 1,

$$\mu = \tfrac{2000}{3005} = 0.665 \text{ sec}$$

From Eq. 9,

$$\theta = \frac{2.68}{0.665} = 4.02$$

From Eq. 8,

$$\rho = \frac{3005 \times 10.78}{2 \times 32.2 \times 200} = 2.514$$

From Fig. 5,

$$Z^2 = 1.87$$

From Eq. 10, the pressure rise is

$$h = 200(1.87 - 1) = 174 \text{ ft}$$

(*b*) *Pressure Drop.* For this installation, find the drop in pressure at the turbine if the penstock flow is increased from zero to 600 cu ft per second

Assume $T = 3.66$ sec for the opening stroke, obtained from an opening time curve similar to that given in Fig 7 for closure.

From Eq. 9,

$$\theta = \frac{3005 \times 3.66}{2 \times 1000} = 5.5$$

From Eq. 8,

$$\rho = \frac{3005 \times 10.78}{2 \times 32.2 \times 200} = 2.514$$

From Fig 6,

$$Z^2 = 0.4$$

From Eq 10, the pressure change is

$$h = 200(0.4 - 1) = -120 \text{ ft}$$

or a pressure drop of 120 ft.

9. Arithmetic Integration Method of Analysis. The water hammer resulting from a gate movement can readily be calculated by using Eq 2 to determine and tabulate the pressure rise (or fall) due to the velocity change during each interval of time of $2L/a$ seconds and by properly combining these tabulated values of pressure rise (or fall) considered as supernormal and subnormal pressure waves. Considering a gate closure and letting ΔV_1 be the velocity change during the first interval, ΔV_2 the velocity change during the second interval, etc., the pressure change during the first interval is

For velocity reduction

$$\Delta h_1 = +\frac{a}{g}\Delta V_1 \tag{12a}$$

For velocity increase

$$\Delta h_1 = -\frac{a}{g}\Delta V_1 \tag{12b}$$

and the pressure change existing at the end of the nth interval is

For velocity reduction

$$h_n = h_{n-1} + \Delta h_n - 2\Delta h_{n-1} + 2\Delta h_{n-2} - 2\Delta h_{n-3}, \text{ etc} \tag{13a}$$

For velocity increase

$$h_n = -h_{n-1} - \Delta h_n + 2\Delta h_{n-1} - 2\Delta h_{n-2} + 2\Delta h_{n-3}, \text{ etc.} \tag{13b}$$

The penstock velocity at any particular gate opening can be expressed by an equation in the form of one representing the velocity through an orifice as follows:

$$V = B\sqrt{H} \tag{14}$$

From these relations the ordinary water-hammer problem can be solved by means of arithmetic integration To illustrate the method, the solution of the following problem is presented.

10. Example by Arithmetic Integration. Assume the conditions given in Section 8.

(*a*) *Pressure Rise.* Calculate the maximum pressure rise at the turbine for a gate closure to zero as shown in Fig. 7.

From Section 8*a*, $a = 3005$ and $V_0 = 10.78$. Since, from Fig. 7, the initial discharge is 600 cu ft per second, the effective area of the penstock is $600/10.78 = 55.66$ sq ft.

The time of one interval (see Eq. 1) is $\mu = (2 \times 1000)/3005 = 0.665$ sec. The computation that is shown by Table 1 follows this procedure:

1. The time in terms of intervals and seconds is recorded in the first two columns.

2. The initial gate-opening constant, B, is calculated from Eq. 14, or $B = 10.78/\sqrt{200} = 0.7623$, and recorded in the first line of the third column.

3. The initial values of head and velocity are set down in the fourth and fifth columns respectively.

4. The remaining values of B in the third column are set down, using Fig. 7 as follows. For the value at 0.665 sec at the end of the first interval, find the corresponding discharge of 570 cu ft per second. This corresponds to a velocity of $570/55.66 = 10.24$ ft per second, and, therefore, the value of B at the end of the first interval is $10.24/\sqrt{200} = 0.7242$. The values of B at the end of the second, third, etc., intervals are obtained similarly.

5. The change in velocity during the first interval is then estimated. As there are six intervals in the closure, this velocity change would be not greater than $10.78/6 = 1.797$ ft per second, owing to the slow motion of the gate at the start. Accordingly, the value selected would be somewhere between 0 and 1.797. Estimating this change at 0.161, the pressure rise during the first interval from Eq. 12*a* would be $(3005/32.2) \times 0.161 = 93.32 \times 0.161 = 15.0$ ft This is entered in the sixth column as Δh.

6. The total head at the turbine at the end of the first interval is then calculated as $200 + 15.0 = 215.0$ and entered in the fourth column.

7. The velocity remaining at the end of the first interval is $10.780 - 0.161 = 10.619$, which is entered in the fifth column.

8. The value of B corresponding to a head of 215.0 and a velocity of 10.619 is then calculated from Eq. 14, $B = 10.619/\sqrt{215.0} = 0.724$, and, if this value checks the one already recorded in Col. 3, the estimate of velocity change, i.e., 0.161, is correct. If this value of B does not check the one already recorded, a new estimate of velocity change is then made and the above steps are repeated until a satisfactory check is obtained.

9. The remaining figures in the table are filled out similarly, with h obtained by means of Eq. 13*a*; for example, the h at the end of the third interval is calculated as follows:

$$h_3 = 60.4 + 188.6 - 2 \times 75.4 + 2 \times 15.0 = 128.2$$

TABLE 1

ARITHMETIC INTEGRATION

(1) Time: Intervals	(2) Time: Seconds	(3) Gate Constant, B	(4) Head, H	(5) Velocity, V and ΔV	(6) $\Delta h = 93.32\Delta V$	(7) h
0	0	0.7023	200.0	10.780		0
				0.161	15.0	
1	0.665	.7242	215 0	10.619		15.0
				0.807	75.4	
2	1.33	.6073	260 4	9.812		60.4
				2.020	188.6	
3	2.00	.4306	328.2	7.792		128.2
				3.215	300.0	
4	2 66	.2376	371.8	4.577		171.8
				3.040	284.5	
5	3.33	.0864	312.7	1 528		112.7
				1.528	142.6	
6	4.00	0000	229 9	0.000		29.9

It will be noticed that, for these conditions, the maximum water hammer of 171.8 ft found by the arithmetic-integration method checks the 174.0 ft determined in Section 8*a* by the Allievi charts.

(*b*) *Pressure Drop*. The solution for pressure drop is identical with that for pressure rise except that it employs Eqs. 12*b* and 13*b*.

11. Graphical Method of Analysis. The graphical method of determining water-hammer pressure consists of plotting a series of parabolas representing discharge through the turbine gates during the disturbed regimen as a series of orifices under varying heads and a series of straight lines representing the pressure due to the direct and indirect water-hammer blows. With the zero subscript to indicate initial conditions, and lower-case letters primed to indicate head, velocity, etc., in terms of relative values, the orifice relation is given by

$$v' = \tau'\sqrt{h'} \qquad [15]$$

where

$$v' = \frac{V}{V_0} \quad h' = \frac{H}{H_0} \quad \text{and} \quad \tau' = \frac{B}{B_0} \qquad [16]$$

With the penstock from turbine to forebay assumed uniform in thickness and diameter, the water-hammer equation is given by

$$\begin{aligned} h'_{B_{t_{1/2}}} - h'_{A_{t_1}} &= -2\rho(v'_{B_{t_{1/2}}} - v'_{A_{t_1}}) \\ h'_{A_{t_1}} - h'_{B_{t_{1\frac{1}{2}}}} &= +2\rho(v'_{A_{t_1}} - v'_{B_{t_{1\frac{1}{2}}}}) \end{aligned} \qquad [17]$$

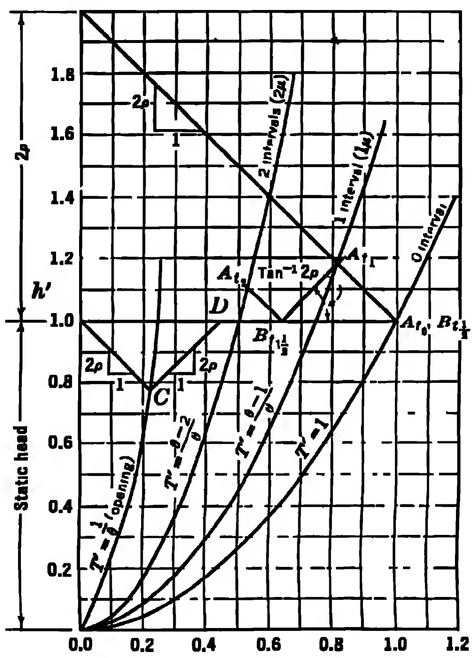
where $H_{A_{t_1}}$ = head, in feet, at the gate end of the penstock at time t_1;

$H_{B_{t_{1/2}}}$ = head, in feet, at the forebay end of the penstock at time $t_{1/2}$ ($\frac{1}{2}$ interval);

$V_{A_{t_1}}$ = velocity, in feet per second, at the gate end of the penstock at time t_1, etc.;

ρ = the parameter as given by Eq. 8.

To solve Eqs. 15 and 17 graphically, a diagram as illustrated by Fig. 8 is constructed. The family of parabolas starting from the origin is first constructed.



Graphical method of solution of water hammer caused by movement of turbine gates

One parabola is made for each interval of time (or fraction of interval). In the diagram the curve $\tau' = 1$ gives the relation between h' and v' for the initial gate opening as in Eq. 15, and the curve $\tau' = 1 - (2L/aT) = 1 - (1/\theta) = (\theta - 1)/\theta$ for the gate opening at the end of the first interval, etc. From the point (1, 1), a line of slope (-2ρ) is drawn which intersects $\tau' = 1 - (1/\theta)$ at A_{t_1}. This line represents the first straight line of Eq. 17. Then a line is drawn through A_{t_1} with a slope of $(+2\rho)$, intersecting the line $h = 1$ at $B_{t_{1\frac{1}{2}}}$, etc. The coordinates of the points A_{t_1}, A_{t_2}, etc., indicate the head and velocity conditions at the turbine end of the penstock at the conclusion of intervals

1, 2, etc., and the coordinates of the points $B_{t_{1/2}}$, $B_{t_{1\frac{1}{2}}}$, etc., show the head and velocity conditions at the forebay end of the penstock at the conclusion of intervals ½, 1½, etc. The pressure-time curve for the turbine end of the penstock is obtained by plotting the values of head at the points A_{t_0}, A_{t_1}, A_{t_2}, etc., against 0, $2L/a$, $4L/a$, etc., seconds, respectively.

If friction is an appreciable quantity in relation to the total head on the turbine, it should be taken into account. This can be done graphically by assuming that the friction acts as though it were concentrated at one or more points along the pipe. The greater the number of points selected for concentrating the friction, the greater the accuracy obtained. Usually it is

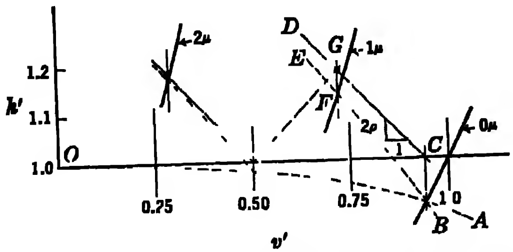


FIG. 9. Graphical method of taking friction into account

sufficiently accurate to assume that all the friction is concentrated at the gate or valve. Figure 9 illustrates the method of taking account of friction on the assumption that it acts at the turbine end of the conduit. The parabolas representing the discharge-head relations for the various gate positions at interval points during the closure are drawn first, and then the line OA is drawn below the X axis by the amount of the friction loss in the conduit. Next, the diagram is begun at point C, which is located on the X axis directly above point B, which is at the intersection of the parabola for the initial gate opening and the friction curve. The line DC is drawn through C with the slope of (-2ρ), and then the line EB is drawn below DC by the amount of the friction loss. This line intersects the parabola for the end of the first interval at F. A vertical line through F intersects DC at point G, which gives the velocity and head conditions at the end of the first interval. The same procedure is followed for succeeding intervals, as shown by the diagram.

12. Example by Graphical Method. Assume the conditions given in Section 8.

(*a*) *Pressure Rise.* Calculate the maximum pressure rise at the turbine for a gate closure to zero as shown in Fig. 7.

From Section 8*a*, $a = 3005$ and $V_0 = 10.78$. The time of one interval (see Eq. 1) is $\mu = (2 \times 1000)/3005 = 0.665$ sec.

From Eq. 9,

$$\theta = \frac{4}{0.665} = 6$$

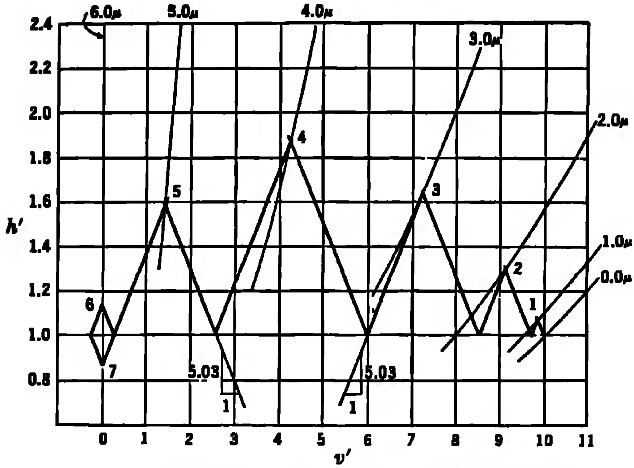


FIG. 10. Graphical solution of pressure rise problem in Section 8.

First the values of τ' (see Eq. 15) are obtained for as many points along the gate curve as desired in order to plot the parabolas shown on Fig. 10 These values are obtained as shown in Table 2 for every interval of time during the closure.

TABLE 2

VALUES OF τ' FOR INTERVALS DURING GATE CLOSURE

Time: Interval	Time: Seconds	Gate Opening, %	Cu Ft per Sec (Fig. 7)	V, ft per sec	τ'
0	0	100.0	600	10.78	1.00
1.0	0.66	87.0	570	10 24	0.949
2.0	1.33	66.8	478	8.59	0.796
3.0	2.00	46.4	339	6.09	0.565
4.0	2.66	25.6	187	3.36	0.311
5.0	3.32	9.4	68	1.22	0.113
6.0	4.00	0	0	0	0

With these values of τ' the family of parabolas as shown can be drawn, and then the straight lines of slope plus and minus

$$2\rho = \frac{3005 \times 10.78}{32.2 \times 200} = 5.026$$

are drawn. The maximum pressure is shown to be

$$1.86 \times 200 = 372 \text{ ft}$$

The maximum pressure rise is 172 ft or 86%. This result checks with the 174.0 ft found in Section 8a by the Allievi charts and the 171.8 ft found in Section 10 by arithmetic integration.

(*b*) *Pressure Drop.* The solution for pressure drop is similar to that for pressure rise. In this case, however, first a line is drawn from the origin with the slope of (-2ρ) intersecting the first τ' line below the static head line as at C in Fig. 8. From point C a straight line with the slope of $(+2\rho)$ is drawn intersecting the line $h = 1$ at D, etc.

13. Critical Governor Time. Although the total time of closure of the ordinary hydraulic turbine is much larger than μ, it is evident that there is a closure from an intermediate gate to zero involving a timing equal to the critical time μ. The pressure rise from such a gate closure is obtained from Eq. 2 and may be greater than the pressure rise occurring when closure from full gate to zero is made. Consequently, it is desirable to investigate also partial closures to zero as well as full gate closures to zero to obtain the maximum pressure rise to which the penstock may be subjected [11].

It should be noted that the percentage of time of governor action is not necessarily proportional to the percentage of gate motion, owing to the fact that certain time elements enter into the problem caused by the physical limitations of the governor itself. In the absence of exact data, however, the time of closure from any initial gate opening can be safely assumed as directly proportional to the time of closure from full gate opening.

14. Pressure Conditions along the Penstock. When, as is always true of hydroelectric plants, the duration of uniform closure is equal to or greater than the critical time $2L/a$, the maximum rise of pressure occurs at the gate, and, if the diameter and thickness of the penstock are constant from there to the forebay, the pressure rise reduces to zero uniformly along the length of the penstock. When the penstock is of varying diameter, it is sufficiently exact to assume for design that the magnitude of the maximum superpressure or depression found at the turbine diminishes from the turbine to zero at the point of open water and has a value at any point, k, equal to that given in the following equation

$$h_k = \frac{h(l_1 v_1 + l_2 v_2 + \cdots l_k v_k)}{LV} \qquad [18]$$

where h_k = the water hammer at point k;
h = the water hammer at the turbine,
V = the effective velocity for the whole pipe from Eq. 7;
L = the total length of the penstock, including the surge-tank riser;
$l_1, l_2, l_3 \cdots l_k$ = successive lengths of penstock, beginning at the forebay, having constant-velocity values of $v_1, v_2, v_3 \cdots v_k$, respectively.

15. Water Hammer and Mechanically Operated Relief Valves. Where the length of penstock between open water or a surge tank and the

turbine is appreciable, a relief valve is sometimes installed. An approximate rule is that when the time of governor action is shorter than

$$\frac{\text{Length of penstock} \times \text{Velocity in penstock}}{10 \times \text{Head}}$$

such a valve is needed. The valve diverts the water from the runner at times of sudden load rejection and reduces the effect of water hammer.

A mechanically operated relief valve is direct-connected to the turbine-gate mechanism. During the period of sudden gate closure there is, for any gate opening and head, a fixed discharge for both the turbine and the valve. All problems of water hammer involving such relief valves can be solved by means of a curve similar to Fig. 7, in which the discharge element includes the discharge through both the turbine and valve under constant head conditions (no length of penstock), and the time element includes both the time of gate closure and the time required for the relief valve to close gradually after the turbine gates shut [8].

16. Water Hammer in the Conduit above the Surge Tank. Surge tanks act as a point of relief and reduce the pressure rise appreciably in the conduit upstream from the tank. The penstock, extending between the tank and the turbine, usually undergoes a considerable pressure change, and the conduit upstream from the surge tank undergoes a lesser pressure change.

Section 14 indicates the method of calculating the water hammer at any point along the penstock. The water hammer at the base of the riser pipe, so computed, travels undiminished up the conduit towards the forebay to a point determined by the following equation, then reduces linearly to zero at the forebay.

$$l_c = \frac{ta_c}{2} \tag{19}$$

where l_c = the distance, in feet, from the forebay to the point in question;
a_c = the velocity of the pressure wave in the conduit, from Eq. 3;
t = the net equivalent time of gate operation (see Section 7) as used for computing the penstock water hammer.

If, as would be quite unusual, l_c from Eq. 19 results in a distance greater than the length of the conduit, the water hammer at the riser pipe can be assumed to reduce uniformly to zero at the forebay. This is a conservative assumption, as, for such cases, the water hammer in both the conduit and the penstock would be reduced.

The computations for a surge-tank installation can be accurately made by the graphical method with three interrelated diagrams: one for the penstock, a second for the tank, and the third for the pipeline [5]. A method giving approximate results is the use of the equations devised by Calame and Gaden [12, 13, 14].

17. Allowance for Approximations. The allowance necessary to compensate for possible inaccuracies in computed water hammer depends on the nature of the installation. Allievi's theory of water hammer is correct, but the curves and equations in this chapter, which are based on Allievi's theory, are correct for the following conditions only:

(*a*) Diameter and thickness of the pipe is constant.
(*b*) The gate-closure time, *t*, is uniform.
(*c*) The upper end of the pipe is at open water.

If approximations are made in computing water hammer under more complicated conditions, allowance must be made for possible errors. It is therefore recommended that water-hammer values in excess of or below static, computed under such circumstances, be multiplied by the following factors, *x*.

	Use of Charts	Arithmetic Integration or Graphical Method
1. Penstock of constant diameter and thickness	$x = 1.05$	$x = 1.0$
2. Penstock of variable diameter and variable thickness	$x = 1.10$	$x = 1.05$

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CHAPTER 35

SURGE TANKS

A General

1. Theory. Water-hammer pressure, the theory of which is treated in Chapter 34, is created in long closed conduits by sudden closure of the turbine gates The water-hammer pressure provides the necessary force to retard the flow in the conduit when load is rejected by the turbine For very long conduits, the water hammer corresponding to normal operation of the turbine may be very great and may require extraordinary strength of the conduit to withstand it, and the violent fluctuations of pressure in the con-

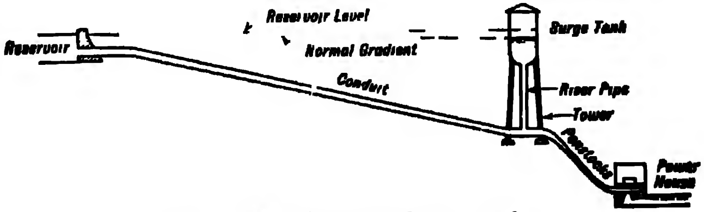

Fig 1 Usual location of surge tank

duit may seriously interfere with proper turbine regulation For sudden opening of the gates, the resulting negative water hammer, or reduction of pressure, provides the necessary force to accelerate the water and is correspondingly objectionable for very long conduits on account of difficult turbine regulation

The simplest means of eliminating positive water-hammer pressure is to provide a by-pass to take the rejected flow This may be accomplished by installing a relief valve* at the turbine or, as shown in Fig 1, a surge tank at the lower end of the conduit The relief valve is very effective for gate closure in that it provides a place for the surplus water to go; but for gate opening, it naturally cannot supply the necessary water demanded by the turbine, and consequently it has no effect on negative water hammer For these reasons, a surge tank is invariably provided at the lower end of all very long closed conduits

In the case of a simple surge tank, shown in Sketches *A* and *B* of Fig 2, the water simply flows into the tank when rejected by the turbine As the

* Described in Chapter 39, Section 17 (Pressure Regulators in Turbines).

water rises in the tank, a retarding head is created, which decreases the conduit velocity. When the velocity in the conduit is reduced to that demanded by the turbine, the water in the tank starts to fall and fluctuates up and down like the swing of a pendulum until damped out by friction. For a sudden increase in load, the additional discharge required by the turbine flows out of the tank, and the consequent lowering of water surface creates an accelerating head which increases the flow in the conduit. When the conduit discharge corresponds to the turbine demand, the water surface in the tank ceases to fall. The action of a simple surge tank after a sudden opening of the turbine gates is indicated in Fig. 3.

Ordinarily, the tank is designed so that the water will not spill over the top under the most drastic condition of load rejection. However, in special low-head installations, where conditions are favorable, the tank is allowed to spill over; but it is usually found more economical to provide a sufficiently large tank than to make the frequently expensive provision to take care of the overflow with safety.

2. The Simple Surge Tank. Sketches *A* and *B* of Fig. 2 show outlines of simple surge tanks. The surge tank is always located as close as possible to the powerhouse, in order to reduce the length of penstock to the minimum, and preferably on high ground, to reduce the height of the tower. If the site of the tank is sufficiently high, the tower shown in Sketch *A* is omitted, the tank rests directly on a concrete foundation containing the conduit, and an orifice of ample size to permit flow in and out of the tank is provided in the top of the conduit.

The conduit accelerating and retarding heads, induced by a change of water surface in the simple tank, accumulate slowly, corresponding to the gradual change of water level in the tank. The action of the simple tank is sluggish as compared with that of other types, and, as will be shown later, the simple tank requires the greatest volume. Hence, except for special cases, it is the most expensive and is seldom adopted in preference to other types.

3. The Restricted-orifice Surge Tank. Sketches *C* and *D* of Fig. 2 show outlines of restricted-orifice surge tanks. The distinctive feature of this type is a restricted orifice installed between the conduit and the tank. The object of the restricted orifice is to create an appreciable friction loss when the water is flowing to or from the tank.

When load is rejected by the turbine, the surplus water passes through the restricted orifice, and immediately a retarding head, equal to the loss due to the restricted orifice, is built up in the conduit. The size of the orifice may be designed for any desired retarding head. If the orifice is large, the tank passes into the simple tank class and the retarding head is negligible. If the orifice is infinitely small, the retarding head is equal to the water hammer in the conduit with no surge tank. The size adopted is usually such that the initial retarding head, for full load rejected by the turbine, is approximately equal to the ultimate rise of water surface in the tank.

The more quickly the accelerating and retarding heads are applied, the more effective will be the surge tank in the adjustment of the conduit discharge; hence less water will have to be stored in or delivered from the tank, and the tank may therefore be smaller.

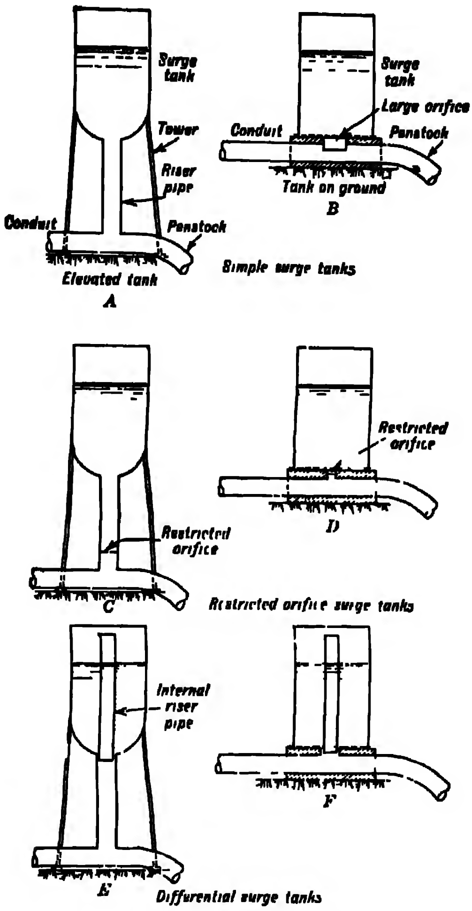

FIG. 2. Types of surge tanks.

Figure 4 shows the action of a restricted-orifice tank for an increase in load, and Fig. 6 shows a comparison of the restricted-orifice tank and the simple tank. Figure 4 shows that, when the turbine gates are opened, an accelerating head in the conduit is immediately created The water in the tank gradually lowers, and, at the end of the first quarter cycle (85 sec), the pressure in the conduit corresponds to the level in the tank and the flow out of the tank has been reduced to zero.

The restricted-orifice tank offers greater efficiency and economy than the simple tank; but the desirable sudden creation of accelerating and retarding head in the conduit also induces correspondingly sudden fluctuations of head on the turbine, which the governors may have difficulty in accommodating. The simple tank is ideal, as far as ease of governing is concerned, in that the

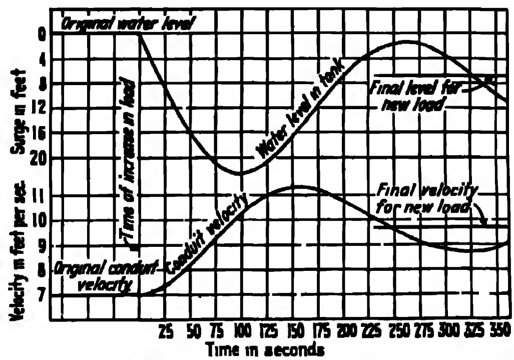

FIG. 3. Action of a simple surge tank.*

head changes are so gradual that even a very slow-acting governor has no difficulty in following the change of pressure. On account of the sudden pressure changes in restricted-orifice tanks, this type cannot be adopted for many installations where close governing is required and where the cost of the necessary additional inertia of the rotating elements of the generating units, to compensate for such fluctuations, would be prohibitive.

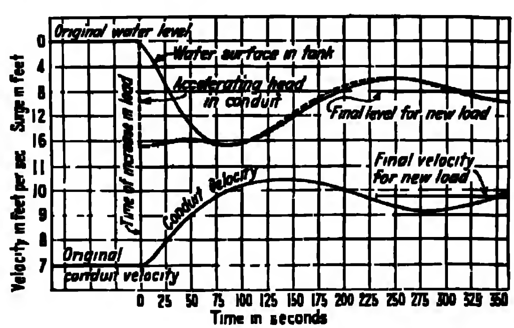

FIG. 4. Approximate action of a restricted-orifice surge tank.

4. The Differential Surge Tank. Sketches *E* and *F* of Fig. 2 show outlines of the differential surge tank. This type is designed to provide a compromise between the simple tank and the restricted-orifice tank. It differs from the simple tank only in having the additional internal riser, as shown

* See p. 789 of Ref. 2 of Section 16.

The internal riser is of smaller diameter than the connection to the conduit. At the base of the internal riser there is an annular port, communicating with the tank. The area of this port is proportioned to suit the conditions under which the tank is to operate. The characteristics of the surge depend upon the area of the port.

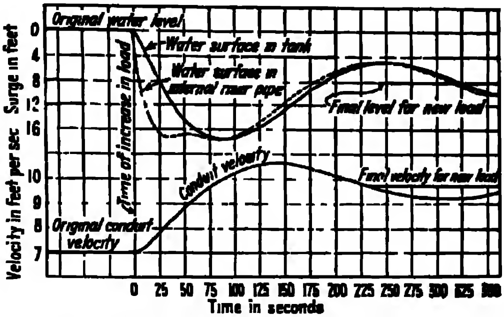

FIG. 5. Action of a differential surge tank.*

In the differential tank, operating for gate opening, the water first falls in the internal riser, establishing in a short time a relatively large accelerating head on the conduit. The level of the tank falls slowly, supplying the demanded increment of flow through the ports at the base of the riser. When the gates are closed, water rises in the internal riser, establishing a retarding

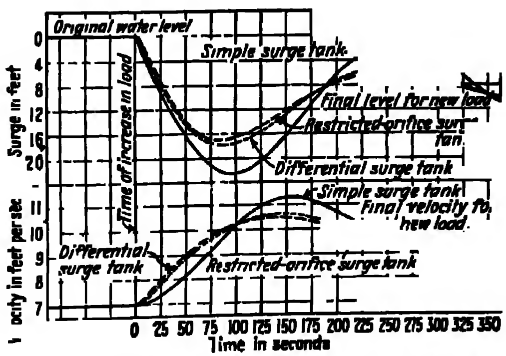

FIG. 6. Comparison of action of different surge-tank types.

head on the conduit as well as a differential head on the port, which forces the water rejected by the turbines through the port into the tank.

The action of the differential tank for an increase in load is shown in Fig. 5. Comparison of the action of this type of tank with that of a simple and a restricted-orifice tank is shown in Fig. 6.

* See p. 789 of Ref. 2 of Section 16.

A comparison of Figs. 4 and 5 shows that the action of the differential tank is similar to that of the restricted-orifice tank, except that the initial pressure change and head on the turbine, instead of occurring instantly as in the restricted-orifice tank, or very slowly as in the simple tank, occurs quickly enough for good efficiency of the tank and is still spread over a period long enough to enable the governors to adjust the turbine gates to compensate for the change in head.

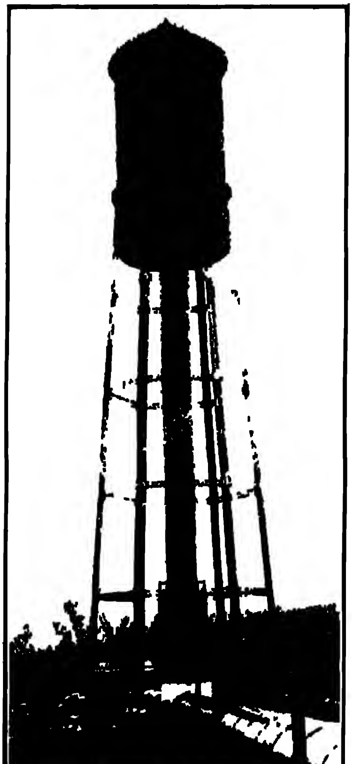

FIG. 7. Forty-foot differential tank 245 ft high at Browns Falls plant of Northern New York Utilities, Inc.

5. Design of Surge Tanks. The theory of regulation of conduit flow by the use of surge tanks is quite involved, being, in fact, too intricate for proper treatment within the scope of this book. Correct design is so important to successful operation that the data should be very carefully analyzed by an engineer who has devoted much time to the study of the theoretical considerations and who has had the rare opportunity of applying the theory extensively in practice. For an exact analysis of the problem, involving also tanks of special shapes, the reader may consult Ref. 3 of Section 16.

However, simple methods have been devised, by which the approximate dimensions of a surge tank can be obtained for purposes of preliminary estimates, economy studies, and other obvious uses. Such methods should never be used for final designs.

Figure 8 is a diagram made by R. D. Johnson for the approximate solution of surge-tank problems. The diagram is made for both differential and simple tanks. Johnson states that the curves for the simple tank are not extremely accurate, and later investigations indicate that some or possibly all of them should be slightly raised for the larger values of K_a'. It is likely, for example, that the top curve would pass through the point whose coordinates are 141.5 and 87.0 instead of 141.5 and 89.5 Note that the maximum possible value of N_r' is 79.6 and occurs when K_r' is 63.3. The diagram is used in the following manner:

In the following nomenclature, the units of measure are the foot and second.

Let

A = area of the conduit;
R = area of the internal riser of a differential surge tank;
L = length of the conduit from forebay to surge tank;
F = net area of surge tank, i.e., area in excess of internal riser area;
f_a = total loss of head in conduit from reservoir to surge tank, including velocity head in the conduit at the surge tank riser,* assumed for accelerating conditions;
f_r = same for retarding conditions;
H = net head on the turbines, used for test of incipient stability;
d_a = lowest draft on pond, assumed for accelerating conditions;
d_r = highest flood height on crest, assumed for retarding conditions;
y_a = fall of water level in the tank from its initial position previous to a load increase;
y_r = rise of water level in the tank from its initial position previous to a load decrease;
Y_a = maximum fall of water level in the tank below crest of dam;
Y_r = maximum rise of water level in the tank above crest of dam;
c = friction characteristic of the conduit, or $cv^2 = f$;
K_a' and N_a' = constants for accelerating conditions;
K_r' and N_r' = constants for retarding conditions;
v_1 = initial general conduit velocity before acceleration begins;
v_2 = initial general conduit velocity before retardation begins;
v_c = critical conduit velocity;
g = acceleration of gravity = 32.2;
p = percentage of velocity change.

Since friction in the conduit decreases the upward surges for retarding conduit-velocity conditions and increases the downward surges for accelerating conditions, the value of c to be adopted should correspond to the minimum conceivable friction for retarding conditions and the maximum conceivable friction for accelerating conditions. Possible variations in friction for various types of conduits are given in Section 10 of Chapter 8.

In determining the maximum discharge of the turbines, it should be remembered that the guarantees of the manufacturers are sometimes exceeded. Therefore, a margin of safety should be applied when computing the maximum conduit velocity.

It is customary to design the surge tank to accommodate full load rejected by the turbines, such as might occur after a short circuit on the transmission line. The choice of the load increase for which the tank should be designed depends on the number of turbines in the plant, the nature of the power market, and other considerations. It is possible for the operators to throw full load on the plant suddenly; but this is quite unlikely to be a normal operating condition, particularly if more than one turbine is installed. It is cus-

* There may be a Venturi at the riser.

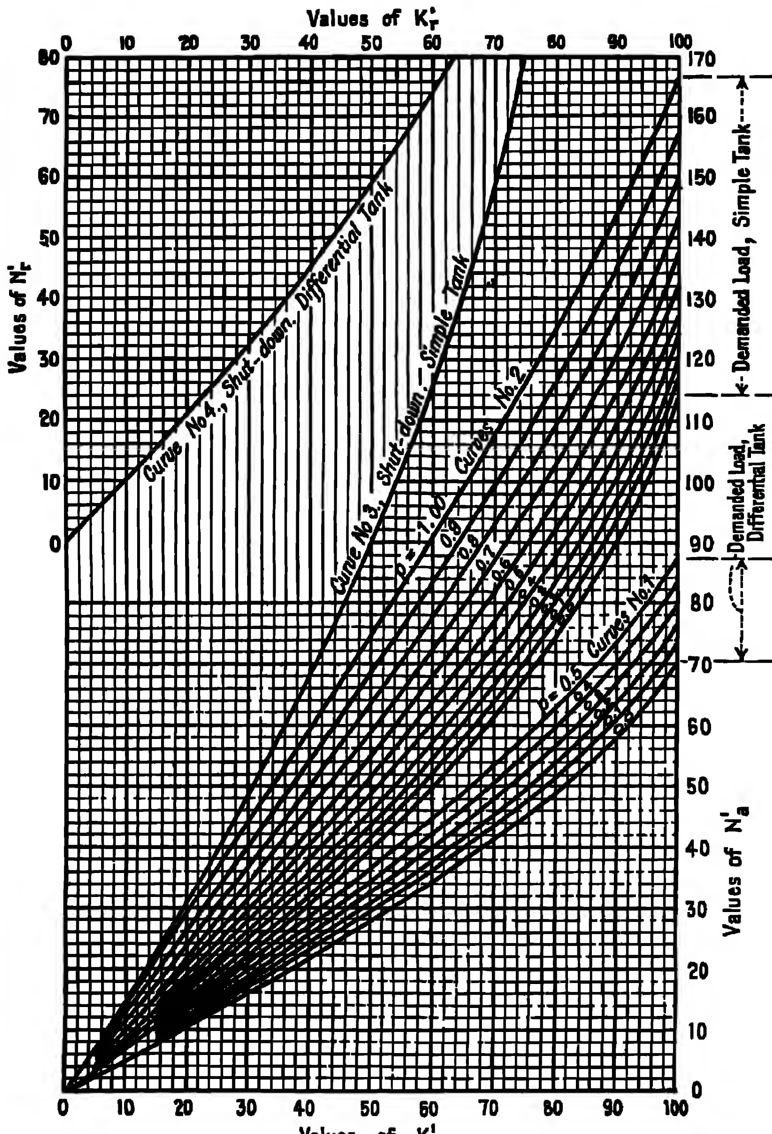

FIG. 8 Johnson surge-tank diagram.

tomary to adopt a load change corresponding to that which may be expected from normal power-market demands. On large systems, the sudden changes of load demand are insignificant, but for an individual plant carrying a widely fluctuating load the percentage load increase may be quite severe. For plants of more than one unit, supplying fairly large systems, an increase of load from three-quarters to full is a frequent assumption.

The various steps necessary for the use of the diagram, and an explanatory example, will now be given.

Acceleration

Step 1. Compute the minimum possible full gate discharge, Q, of the turbines and the corresponding v_2 in the conduit under lowest headwater conditions for the value of f_a in the next step.

Step 2. Compute the maximum possible friction loss, f_a, in the conduit, for the computed value of v_2 in the first step. Obviously, these two steps are interdependent and must be solved by trial.

Step 3. Compute c from

$$c = \frac{f_a}{v_2^2} \qquad [1]$$

Step 4. Compute F in terms of N_a' from

$$F = \left(\frac{N_a'}{100cv_2\sqrt{2g/AL}}\right)^2 \qquad [2]$$

Step 5. Assume a velocity change from v_1 to v_2 for which the surge tank is to be designed.

Step 6. Compute K_a' in terms of y_a from

$$K_a' = \frac{100c(v_2^2 - v_1^2)}{y_a} \qquad [3]$$

Step 7. Assume several values of surge, y_a, as indicated in Table 1.

Step 8. For each surge, compute the corresponding K_a' from Eq. 3 of Step 6.

Step 9. Compute p from

$$p = \frac{v_2 - v_1}{v_2} \qquad [4]$$

Step 10. With K_a' and p from Steps 8 and 9, find the corresponding N_a' from Curves 1 or 2 of the diagram of Fig. 8. Figure 8 has been constructed for a maximum value of $p = 0.5$ of load thrown on, since that value is well outside usual operating conditions. For a greater value of load thrown on, see Fig. 1, p. 782, or Ref. 2, Section 16, in which $K = 100/K_a'$ and $N_a = N_a'$.

Step 11. With this value of N_a', compute the corresponding value of F from Eq. 2 of Step 4.

Step 12. Compute the departure of the original friction gradient from crest level, from

$$\text{Departure} = cv_1^2 + d_a$$

Step 13. This departure, added to the assumed surge, y_a, in Step 7, gives the lowest surge, Y_a, below crest level.

Step 14. Compute, from the value of F derived in Step 11, the corresponding tank diameter from

$$D = \sqrt{\frac{F + R}{0.785}} \tag{5}$$

The area, R, of the internal riser may be assumed to be the same as that of the conduit, which is an approximation on the safe side, as the riser is usually slightly smaller than the conduit.

Step 15. Values of Y_a and D from Steps 13 and 14, as shown in Table 1, may now be plotted in the form of a curve as in Fig. 9.

Retardation

Step 1. Compute the maximum possible full gate discharge, Q, of the turbines, and the corresponding velocity, v_2, in the conduit, under highest headwater conditions, for the value of f_r in the next step. Note that this value of Q is larger than that for acceleration because the original net head is greater.

Step 2. Compute the minimum possible friction loss, f_r, in the conduit for the computed value of v_2 in the first step.

Step 3. Compute c from

$$c = \frac{f_r}{v_2^2}$$

Step 4. Compute N_r' in terms of F, from

$$N_r' = \left(100cv_2\sqrt{\frac{2g}{AL}}\right)\sqrt{F} \tag{6}$$

Step 5. Compute y_r in terms of K_r', from

$$y_r = \frac{100cv_2^2}{K_r'} \tag{7}$$

Step 6. Assume several values of surge-tank area, F, as indicated in Table 2.

Step 7. For each area, compute the corresponding N_r' from Eq. 6.

Step 8. With this value of N_r', find the corresponding value of K_r' from Curve 3 or 4 of the diagram of Fig. 8.

For Curve 3, read values of N_r' in the right margin of Fig. 8. For Curve 4, the maximum $K_r' = 63.5$ with $N_r' = 78.6$.

Step 9. With this value of K_r', find the corresponding value of y_r from Eq. 7.

Step 10. Compute the departure of the original friction gradient from crest level, from

$$\text{Departure} = d_r - cv_2^2$$

Step 11. This departure, added algebraically to the value of y_r from Step 9, gives the highest surge, Y_r, above crest level.

Step 12. Compute, from the assumed values of F in Step 6, the corresponding tank diameter from Eq. 5.

Step 13. Values of Y, and D from Steps 11 and 12 as shown in Table 2, may now be plotted in the form of a curve as in Fig. 9.

Ordinates between the two curves of Fig. 9 show the required height of the tank for any adopted value of diameter. However, the curves should now be tested for "incipient stability" and "critical velocity."

TEST FOR INCIPIENT STABILITY

To insure incipient stability, that is, steady conditions under slight load changes, Thoma [1] has shown that the area of the tank should not be less than

$$F_M = \frac{AL}{2gcH} \qquad [8]$$

Or, in terms of diameter, the diameter should not be less than

$$D_M = \sqrt{\frac{R + (AL/2gcH)}{0.785}} \qquad [9]$$

Simplifying,

$$D_M = 1.13\sqrt{R + \frac{AL}{2gcH}} \qquad [10]$$

The value of c to be used in the above equations should be the c adopted for retardation. The value of H to be used should be the smallest possible net head under full load conditions.

Equations 8, 9, and 10 are theoretical expressions embodying the assumption of constant efficiency of turbine. Johnson has shown* that, for operation on the drooping side of the efficiency curve, the tank dimensions used must be still larger. However, as the shape of the efficiency curve is not usually known for preliminary investigations, the approximate diameter of the tank, to insure incipient stability, may be assumed 25% larger for the differential tank and 40% larger for the simple tank.

TEST FOR CRITICAL VELOCITY

For a given area of tank, there is a critical velocity which, if reduced to zero, will give the maximum height of surge. Johnson's equation for the critical velocity is

$$v_c = \frac{0.1}{c}\sqrt{\frac{AL}{F}} \qquad [11]$$

Or, in terms of diameter,

$$v_c = \frac{0.1}{c}\sqrt{\frac{AL}{0.785D^2 - R}} \qquad [12]$$

* See p. 269 of Ref. 5, Section 16.

The value of c that was adopted for retardation should be used.

The critical velocity may be greater or less than the full-load velocity, v_2. If the critical velocity is greater than v_2, no correction need be made in the curves of Fig. 9, except that, if the critical velocity is only slightly greater than v_2, it might be thought advisable to design for the critical velocity, than which no velocity could give a greater surge. If the critical velocity is less than v_2, it should be used for determining a revised surge to fix the height of the tank of the given size.

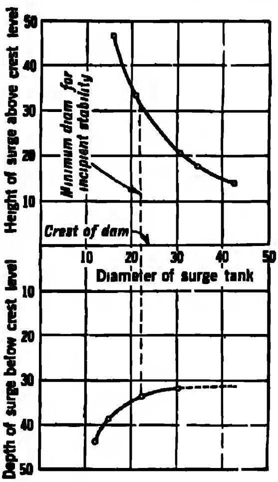

FIG. 9. Diagram for explanatory example differential surge tank.

ADOPTED SIZE OF TANK

From Fig. 9 it is seen that the diameter varies inversely as the height. Consequently, any combination of diameter and height that will correspond to the curves of Fig. 9 will fulfil the requirements. The combination adopted should be such as to make the cost of the tank the minimum, it being borne in mind that the tank should be as large in diameter as possible, consistent with economy, to limit the extent of the surges for which the governor must compensate.

6. Explanatory Example. The following example, for the approximate design of a differential surge tank, explains the foregoing method.

ACCELERATION

Step 1. Let

$$Q = 292 \text{ sec-ft}$$

$$v_2 = 6.60 \text{ ft per second}$$

Step 2. Let

$$f_a = 18.4 \text{ ft}$$

Step 3. Compute

$$c = \frac{18.4}{6.6^2} = 0.422$$

Step 4. Let

$$A = 44.18 \text{ sq ft}$$

Let

$$L = 12{,}800 \text{ ft}$$

Compute

$$F = \left(\frac{V_2}{100 \times 0.422 \times 6.6 \sqrt{\frac{2 \times 32.2}{44.18 \times 12,800}}}\right)^2$$

or

$$F = \left(\frac{V}{2.98}\right)^2$$

Step 5. Let Q change from 219 to 292. The corresponding velocity change is

$$v_2 = 6.6$$

$$v_1 = 4.95$$

Step 6. Compute

$$K_a' = \frac{100 \times 0.422(6.6^2 - 4.95^2)}{y_a}$$

or

$$K_a' = \frac{804}{y}$$

TABLE I

Step 7. Assume values of surges y_a	8.1	10.0	15.0	20.0
Step 8. Compute K_a from Step 6	100.0	81.0	54.0	40.5
Step 9. Compute $p = \frac{6.6 - 4.95}{6.6} = 0.25$				
Step 10. Find X from Curves No. 1 of Fig. 8	78.0	56.0	34.5	25.0
Step 11. Compute Y from Step 4	685.0	353.0	131.0	70.1
Step 12. Assume $d = 13.5$ ft. Compute departure $= 0.422 \times 4.95^2 + 13.5 =$	23.8	23.8	23.8	23.8
Step 13. Lowest surge* Y = Step 12 + Step 7	31.9	33.8	38.8	43.8
Step 14. Assume $R = A = 44.18$ sq ft. Compute $D = \sqrt{\frac{Y + 44.18}{0.785}}$	30.5	22.5	15.1	12.1
Step 15. Plot Steps 13 and 14 in Fig. 9				

* *Low cut*

RETARDATION

Step 1. Let

$$Q = 304 \text{ sec-ft}$$

$$v_2 = 6.87 \text{ ft per second}$$

Step 2. Let

$$f = 11.0 \text{ ft}$$

Step 3. Compute

$$c = \frac{14.0}{6.87^2} = 0.296$$

Step 4. Let, as before,

$$A = 44.18 \text{ sq ft}$$

$$L = 12{,}800 \text{ ft}$$

Compute

$$K_r' = \left(100 \times 0.296 \times 6.87 \sqrt{\frac{2 \times 32.2}{44.18 \times 12{,}800}}\right) \sqrt{F}$$

or

$$K_r' = 2.17\sqrt{F}$$

Step 5. For complete shutdown, compute

$$y_r = \frac{100 \times 0.296 \times 6.87^2}{K_r'}$$

or

$$y_r = \frac{1400}{K_r'}$$

TABLE 2

Step 6. Assume values of F	160	300	700	900	1360
Step 7. Compute K_r' from Step 4	27.5	37.5	57.3	65.1	80.0
Step 8. Find K_r' from Curve No. 1 of Fig. 8	25.5	34.0	48.5	55.0	63.5
Step 9. Compute y_r from Step 5	54.8	41.2	28.9	25.5	22.0
Step 10. Assume $d_r = 6.0$ ft. Compute departure $= \dfrac{6.0}{0.296 \times 6.87^2}$	8.0	8.0	8.0	8.0	8.0
Step 11. Highest surge, Y_r = Step 10 + Step 9	46.8	33.2	20.9	17.5	14.0
Step 12. Assume $R = A = 44.18$ sq ft. Compute $D = \sqrt{\dfrac{F + 44.18}{0.785}}$	16.1	21.0	30.8	34.7	42.3
Step 13. Plot Steps 11 and 12 in Fig. 9.					

Test for Incipient Stability

The minimum allowed diameter for incipient stability is computed from Eq. 9. Let

$$R = A = 44.18 \text{ sq ft}$$

$$L = 12{,}800 \text{ ft}$$

$$H = 150 \text{ ft}$$

$$c = 0.296 \text{ for retardation as before.}$$

Then

$$D_M = \sqrt{\frac{44.18 + \dfrac{44.18 \times 12{,}800}{2 \times 32.2 \times 0.296 \times 150}}{0.785}} = 17.6 \text{ ft}$$

And as this is to be a differential tank use 25% greater diameter, or 1.25 × 17.6 = 22.0 ft. This is indicated in Fig. 9.

Test for Critical Velocity

The critical velocity is found from Eq. 12, or

$$v_c = \frac{0.1}{0.296}\sqrt{\frac{44.18 \times 12{,}800}{0.785D - 44.18}}$$

$$v_c = \sqrt{\frac{82{,}300}{D - 56.3}}$$

Let

D =	15.00	20.00	25.00	35.00	42.40	45.00
v from previous equation, =	22.10	15.16	12.03	8.40	6.87	6.47

From these calculations it is seen that as the v_c used for retardation was 6.87 no correction need be made to the curve of Fig. 9 for adopted diameters less than 42.4 ft, but if it is desired to adopt a diameter greater than 42.4 ft, the critical velocity must be used to determine the upward surge. For instance, if it is desired to adopt a diameter of 45 ft the surge must be computed from the critical velocity of 6.47 instead of the velocity of 6.87 previously used, the velocity of 6.47 being critical will give the greater surge.

A diameter of 30 ft will be adopted for the tank, and the curves of Fig. 9 show that the top of the tank must be at least 21.5 ft above crest level and the bottom 32.0 ft below crest level making the total height of the tank 53.5 ft.

7. Surges in the Conduit. The excess pressure above crest level in the conduit due to surges in the tank will be equal to Y_r between the surge tank and a critical point on the conduit line and thence will decrease linearly to zero at the forebay. This rule applies to a conduit of constant characteristics throughout but is near enough for practical purposes if the area of the conduit increases towards the forebay.

To determine the location of the critical point on the conduit line let

- l = the distance in feet from the forebay to the critical point
- a = the velocity of wave propagation in the conduit in feet per second
- t = time in seconds required for the excess pressure Y_r to accumulate at the surge tank.

Then

$$l = \frac{ta}{2} \qquad [13]$$

The determination of the value of velocity of wave propagation, a, is given in Chapter 34.

The time, t, may be assumed as follows:

For a simple tank, t is so large that the distance l will be greater than the length of the conduit; hence, for such results, the excess pressure may be assumed to diminish linearly from the surge tank to the forebay; i.e., the critical point is at the surge tank.

For a restricted-orifice tank, t may be taken equal to the time required to close the turbine gates. The critical point will be closer to the forebay for this type of tank than for any other.

For a differential tank, t may be obtained from the following equation:

$$t = \frac{Y_r R}{v_2 A} + \frac{t'}{2} \tag{14}$$

where t' is the time required to close the turbine gates and the other letters are as indicated in Section 5, v_2 being used for retardation. It should be remembered that R, the area of the internal riser, may be smaller than the area, A, of the conduit in special cases. However, R may be assumed to be about 90% of A for preliminary calculations.

The excess pressure in the conduit, due to surges in the tank, may occur simultaneously with the water hammer caused by the sudden acceleration of water in the riser, as explained in Chapter 34; but this does not always happen. However, it is best to add these two effects in determining the maximum excess pressure for which the conduit is to be designed.

The foregoing method applies also to the determinations of subnormal pressures in the tank for load increases. Care should be taken that the pressure in the conduit never becomes negative as, under such conditions, the conduit is likely to collapse.

8. Heating Surge Tanks. In cold climates it is necessary to apply heat to the water to prevent freezing. The diagram of Fig. 10* shows the heat loss from the surface of exposed steel tanks in Btu per hour per square foot of surface area per degree (Fahrenheit) temperature difference between tank water and the atmosphere. The temperature of the tank water is the average inside the tank and is usually assumed to be 32 degrees. The surface area is the entire area of water in contact with exposed steel, plus the area of the top surface. The diagram is made for a wind velocity of 12 mi per hr. Add 0.1 Btu for each mile per hour above 12. In determining the temperature difference and the wind velocity, it should be remembered that minimum temperatures for a given locality seldom last for many hours, nor do high wind velocities usually accompany very low temperatures.

In very cold climates it is the general practice to lag surge tanks to assist in the prevention of freezing. The accepted best practice in this respect is to attach clip angles to the wall of the tank and to bolt to these angles nailing strips, so that the lagging proper will be located 6 in. away from the wall of the tank. Such nailing strips are generally cut to conform to the curvature

* Drawn by L. Goff of the Factory Mutual Laboratories, Boston.

of the tank and are placed in a horizontal position. A space of 1.5 to 2.0 in. is provided between the strips and the tank to allow circulation of air. On very large tanks, where it is a simple matter to bend the lagging, the 6-in. nailing strips can be placed vertically. This arrangement is a somewhat simpler means of providing for the transmission of warm air around the tank. The lagging itself invariably consists of two layers of matched 1-in. lumber with building paper between.

It is necessary to provide for the transmission of air into and out of the tank, on account of the changing volume of water. On small tanks, a narrow slot under the eaves of the roof is usually provided; on larger tanks it is customary to install hinged doors, hung at the top, which are free to move in or out for the transmission of air, but which normally hang closed to confine the heat within the housing.

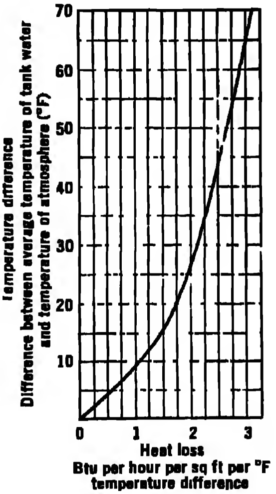

FIG. 10. Loss of heat from exposed steel tanks for wind velocity of 12 mi per hour.

The usual method of heating the tank in cold weather is to use warm-air circulation between the tank and the lagging. The Associated Factory Mutual Fire Insurance Companies Inspection Department recommends, for 2.75-in. wooden tanks and for lagged tanks, one third of the heat units shown in Fig. 10 for exposed steel tanks. However, additional heat units should be allowed for the loss of heat out of the top of the tank. In the case of flap doors and a fairly steady load condition, this loss of heat is not great; but on smaller tanks where a narrow slot is left at the top the loss of heat may be considerable.

Ordinarily, the warm-air heating system depends on natural circulation only, but a more positive means can be installed by using forced ventilation. Other methods of heating include discharging warm water into the tank near the bottom. This can be further augmented by blowing air into the bottom of the tank in order to keep the contents of the tank in circulation. There does not seem to be much difference in the efficiency of the various methods of heating, provided the lagging is installed according to the standard practice. It is necessary to estimate the dissipation of heat through the lagging, and to base the capacity of the heating plant on this figure. Care should be taken, in the case of the natural warm-air circulating system, to see that the air is fairly evenly distributed around the circumference of the tank and for the entire height.

Bubbler systems similar to those described in Section 6g of Chapter 27 have been used successfully in lieu of frostproofing at the Rocky River plant of the Connecticut Light and Power Company, at the Berlin plant of the

Public Service Company of New Hampshire, and at the Cobble Mountain plant of the Western Massachusetts Electric Company.

The matter of fire protection may be important. In the case of an elevated tank, the lagging around the riser near the base of the tank could be made of tile and the wood lagging started well above the ground level. In this way there is little danger of a fire at the ground level spreading to the wood lagging.

Heat is frequently provided at the base of the riser pipe; but, where the riser is of ample size and a slight reduction in area due to an ice coating would not be objectionable, the riser is encased and the heat applied near its top.

Much information regarding heating appliances for tanks is given in "Specifications for Gravity Water Tanks and Steel Towers," published by the Inspection Department, Associated Factory Mutual Fire Insurance Companies, Boston.

For the Browns Falls surge tank, Fig. 7, the riser pipe, which is 11 ft in diameter and about 200 ft high, is inclosed in an octagonal framework, sheathed on the outside with two thicknesses of 1-in. cypress, planed and matched, between which is placed a layer of builder's felt. This framework is made large enough so that a series of stairs are inclosed, leading from a door at the bottom to a floor underneath the tank. No heat is applied to the air space around the riser.

About 3 ft below the bottom of the tank, there is a floor extending out beyond the lines of the tower, and from this floor a double thickness of the same cypress sheathing extends up and around the shell of the tank. This portion of the sheathing is circular in plan and is separated from the steel sides of the tank by an air space of about 8 in. This sheathing is nailed to horizontal cleats bolted to angle irons which were riveted to the shell of the tank when the tank was erected. This sheathing extends up to the eaves of the conical steel roof which is over the tank.

On the platform under the tank and around the top of the riser pipe, there has been placed a rack to which twelve heating units have been attached. Each unit consists of a General Electric Company industrial air heater for 750 degree Fahrenheit operation, Catalog No. 190837G1, Type AH, Form G, 3.8 kw. The units are spaced equally around the riser and inclosed within a sheet-iron partition extending from the floor to the bottom of the surge tank, making an annular space about 30 in wide. No trouble with freezing has been experienced, although the temperature frequently drops to 30 or 40 degrees Fahrenheit below zero.

B. ARITHMETIC INTEGRATION

By Earl B. Strowger *

9. Introduction. In Part A was given a method for determining readily the approximate dimensions of differential surge tanks. The final design of

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the tank and particularly the area of the port (see Section 4) is governed by the same assumptions as in Part A but must be obtained by arithmetic integration, consisting of step-by-step calculations of the changes in hydraulic conditions during small increments of time. The arithmetic integration method will be explained in detail for a differential surge tank only and abstractly for a restricted orifice tank.

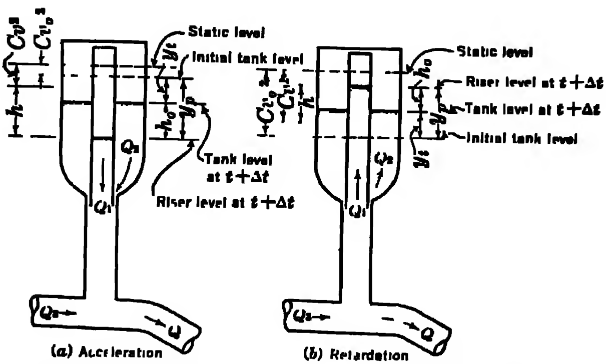

FIG. 11. Differential tank showing surge conditions at any time $t + \Delta t$.

10. Nomenclature. In the following nomenclature, units of feet and seconds are intended. Figure 11 shows some of the symbols as used in connection with a differential tank.

A = area of the conduit between forebay and surge tank;
a = area of surge tank port or orifice;
c_0 = coefficient of discharge of the port;
C = a friction constant such that $Cv^2 = f$ = the loss of head from forebay to surge tank, including velocity head at the surge tank riser, for any velocity, v;
f = loss of head from forebay to surge tank, including velocity head at the surge tank riser, at time, $t + \Delta t$ (see symbol C);
F = net area of the surge tank, i.e., gross area less riser area;
g = acceleration of gravity;
h' = average accelerating or retarding head on the conduit during interval Δt;
h = accelerating or retarding head on the conduit at time $t + \Delta t$;
h_0' = average differential head on the port or orifice during interval Δt;
h_0 = differential head on the port or orifice at time $t + \Delta t$;
L = length of the conduit from forebay to surge tank;
Q = the average rate of discharge passing through the turbine during interval Δt;
Q_1 = the concurrent average rate of discharge supplied or received by the riser pipe;
Q_2 = the concurrent average rate of discharge supplied or received by the surge tank;
Q_3 = the concurrent average rate of discharge supplied or received by the conduit;
R = the area of the riser pipe;
t = a given time;
Δt = an interval of time immediately subsequent to time t;
v' = conduit velocity at time t;
v_0 = original steady conduit velocity;
v = conduit velocity at time $t + \Delta t$;
y_p' = total fall or rise of water level in the riser pipe, below or above the original steady gradient, at time t;

y_p = same for time $t + \Delta t$;
y_t' = total fall or rise of water level in the surge tank below or above the original steady gradient at time t;
y_t = same for time $t + \Delta t$.

11. Fundamental Equations for a Differential Tank. All distances, y, are positive, whether measured downward or upward from original steady gradient. These equations are for a differential surge tank.

Acceleration		Retardation	
$Q_1 = \dfrac{(y_p - y_p')R}{\Delta t}$	[15]	Same	
$Q_2 = c_0 a\sqrt{2gh_0'}$	[16]	Same	
$Q_3 = A\left(v' + \dfrac{h'g\,\Delta t}{2L}\right)$	[17]	$Q_3 = A\left(v' - \dfrac{h'g\,\Delta t}{2L}\right)$	[17*a*]
$Q = Q_1 + Q_2 + Q_3$	[18]	$Q = Q_3 - Q_2 - Q_1$	[18*a*]
$Q_2 = \dfrac{(y_t - y_t')F}{\Delta t}$	[19]	Same	
$v = v' + \dfrac{h'g\,\Delta t}{L}$	[20]	$v = v' - \dfrac{h'g\,\Delta t}{L}$	[20*a*]
$h = (y_p - f) = y_p - C(v^2 - v_0^2)$	[21]	$h = y_p - C(v_0^2 - v^2)$	[21*a*]
$h_0 = y_p - y_t$	[22]	Same	

These equations can be solved if the values of h' and h_0' are known or assumed closely. There are three methods which may be used:

1. Assume h' and h_0' and calculate h and h_0 from Eqs. 21 and 22. Then adopt more accurate values of h' and h_0' and repeat by cut-and-try method. This can be used advantageously at the start until the hydraulic characteristics are plotted. Then use method 2.

2. Plot values of h and h_0, and extrapolate the curve to obtain values of h' and h_0' for the next interval.

3. Assume h' and h_0' for any interval Δt to be equal to h and h_0 at the end of the preceding interval. This method is commonly used.

As indicated in Section 15, a port area must be assumed and sufficient calculations made to determine whether that area is satisfactory. The correct port area is that which will give a total surge in the riser pipe not materially different from the surge in the tank, the latter surge being forecasted by the approximate method described in Part A.

12. Explanatory Example for Acceleration for a Differential Tank. Let full gate turbine discharge be started from zero flow. Assume:

Diameter of tank = 40 ft;
Governor time for full opening = 5 sec;
Minimum forebay level = original steady gradient at surge tank = elevation 2000.00;
R, usually equal to A = area of riser pipe and conduit = 113.1 sq ft;

F = net area of surge tank = 1144 sq ft;
C = maximum possible conduit friction constant = 0.191;
v_o = original steady velocity = 0;
L = length of conduit = 7710 ft;
c_0 = port coefficient = 0.6;
a = area of port = 46.7 sq ft.

The steady full-gate demand at the turbine after the surge has ceased is 1500 sec-ft. For high heads, it may be assumed that the value of Q, for the 5 sec of governor time, increases uniformly from 0 to 1500 sec-ft and remains at 1500 sec-ft thereafter. For low heads, the turbine discharge will vary considerably with the head, and Q should be altered for each step in the calculations to correspond to the head measured to the elevation of water in the riser pipe during that step.

Taking Δt at 1 sec, the computations leading from a time t, of 9 sec to a time $t + \Delta t$ of 10 sec are as follows:

Previous step-by-step calculations have resulted in the following values at time t of 9 sec and the assumed average values for the next interval, Δt, of 1 sec.

Time t	Average for Interval Δt
$y_p' = 32.50$	$h' = 32.46$
$y_t' = 5.30$	$h_0' = 27.20$
$v' = 0.48$	

From Eq. 15,

$$Q_1 = \frac{(y_p - 32.50)113.1}{1} = 113.1\, y_p - 3675$$

From Eq. 16,

$$Q_2 = 0.6 \times 46.7\sqrt{2 \times 32.2 \times 27.20} = 1170$$

From Eq. 17,

$$Q_3 = 113.1\left(0.48 + \frac{32.46 \times 32.2 \times 1}{2 \times 7710}\right) = 62$$

From Eq. 18,

$$1500 = 113.1\, y_p - 3675 + 1170 + 62$$

from which

$$y_p = 34.80$$

and

$$\text{W.S. elevation in riser} = 2000.00 - 34.80 = 1965.20$$

From Eq. 19,

$$1170 = \frac{(y_t - 5.30)1144}{1}$$

from which

$$y_t = 6.31$$

and

$$\text{W.S. elevation in tank} = 2000.00 - 6.31 = 1993.69$$

From Eq. 20,

$$v = 0.48 + \frac{32.46 \times 32.2 \times 1}{7710} = 0.615$$

From Eq. 21,

$$h = 34.80 - 0.191[(0.615)^2 - 0] = 34.73$$

From Eq. 22,

$$h_0 = 34.80 - 6.31 = 28.49$$

This completes the steps for the interval Δt, and the values so obtained are used for time t in the next interval.

13. Explanatory Example for Retardation for a Differential Tank. Let full-gate turbine discharge be reduced to zero flow.

In addition to the assumptions in the previous example, assume:

Governor time for full closing = 5 sec;
Maximum forebay level = elevation 2010.00;
C' = minimum possible conduit friction constant = 0.1274;
v_o = original steady velocity = 13.881 ft per second;
f = original steady conduit friction = $0.1274 \times (13.881)^2 = 24.55$ ft;
Original steady gradient at riser pipe = 2010.0 − 24.55 = elevation 1985.45.

The steady full-gate demand at the turbine before closure is 1570 sec-ft, subject to the same conditions as explained for the acceleration example in Section 12.

Taking Δt = 1 sec, the computations leading from a time t of 9 sec to a time $t + \Delta t$ of 10 sec are as follows.

Previous step-by-step calculations have resulted in the following values at time t of 9 sec and the assumed average values for the next interval Δt of 1 sec:

TIME t	AVERAGE FOR INTERVAL Δt
$y_p' = 34.60$	$h' = 32.83$
$y_t' = 5.35$	$h_0' = 29.25$
$v' = 13.38$	

From Eq. 15,

$$Q_1 = \frac{(y_p - 34.60)113.1}{1} = 113.1\, y_p - 3915$$

From Eq. 16,

$$Q_2 = 0.6 \times 46.7\sqrt{2 \times 32.2 \times 29.25} = 1214$$

From Eq. 17a,

$$Q_s = 113.1\left(13.38 - \frac{32.83 \times 32.2 \times 1}{2 \times 7710}\right) = 1506$$

From Eq. 18a,

$$0 = 1506 - 1214 - (113.1\ y_p - 3915)$$

from which

$$y_p = 37.18$$

and

$$\text{W.S. elevation in riser} = 1985.45 + 37.18 = 2022.63$$

From Eq. 19,

$$1214 = \frac{(y_t - 5.35)1144}{1}$$

from which

$$y_t = 6.42$$

and

$$\text{W.S. elevation in tank} = 1985.45 + 6.42 = 1991.87$$

From Eq. 20a,

$$v = 13.38 - \frac{32.83 \times 32.2 \times 1}{7710} = 13.24$$

From Eq. 21a,

$$h = 37.18 - 0.1274[(13.881)^2 - (13.24)^2] = 34.99$$

From Eq. 22,

$$h_0 = 37.18 - 6.42 = 30.76$$

This completes the steps for the interval Δt, and the values so obtained are used for time t in the next interval.

14. Fundamental Equations for a Restricted-orifice Tank. The same symbols will be used for the restricted-orifice tank as those already used

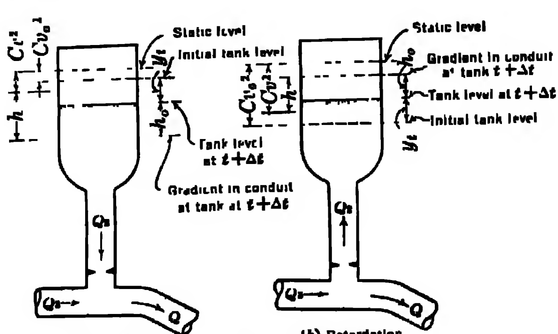

FIG. 12. Restricted-orifice tank showing surge conditions at any time $t + \Delta t$.

for the differential tank. As can be seen by reference to Fig. 12, however, the riser area becomes zero, so that Q_1 and y_p disappear. All distances, y, are positive whether measured downward or upward from original steady gradient.

ACCELERATION		RETARDATION	
$Q_2 = \frac{(y_t - y_t')F}{\Delta t}$	[23]	Same	
$Q_3 = A\left(v' + \frac{h'g\,\Delta t}{2L}\right)$	[24]	$Q_3 = A\left(v' - \frac{h'g\,\Delta t}{2L}\right)$	[24a]
$Q = Q_2 + Q_3$	[25]	$Q = Q_3 - Q_2$	[25a]
$v = v' + \frac{h'g\,\Delta t}{L}$	[26]	$v = v' - \frac{h'g\,\Delta t}{L}$	[26a]
$h_0 = \frac{Q^2}{c_0^2a^2 2g}$	[27]	Same	
$h = h_0 + y_t - C(v^2 - v_0^2)$	[28]	$h = h_0 + y_t - C(v_0^2 - v^2)$	[28a]

Equations 23, 24, and 25 are treated as simultaneous equations for determining the value of y_t, which in turn determines the elevation of the water level in the tank. Then Eqs. 26, 27, and 28 determine the values of v, h_0, and h respectively.

15. General. Computations similar to the above must be carried through from the beginning of the surge transient to the end of one quarter of the surge cycle, i.e., to the point where tank level and riser level become the same. During the period at the start of the transient when y_p changes rapidly, a small value, such as 1 second, should be chosen for Δt. Later, a 5-sec or possibly 10-sec interval may be used without appreciable error. Also, several values of port area should be investigated and the one producing the least number of undulations in the riser water level should be used.

Dead-beat tanks are not economical. They are used only when it is important to limit the amount of undulatory motion in the tank itself on account of some special regulating in connection with a small system or an isolated load. If dead-beat action was required in the case of the 40-ft-diameter tank in the above explanatory example, it would be necessary to increase the diameter to 60 ft to obtain such action.

An accurate check on the work of arithmetic integration applied to accelerating conditions is to plot the accelerating head-time curve (Eq. 21) and to measure the area under this curve out to the point where the velocity of the conduit becomes equal to the demanded velocity. This area, in foot-second units, should equal the momentum of the water in the conduit at the end of acceleration, i.e., equal to Lv/g in foot-seconds, where v is the final conduit velocity. In the case of retardation, the retarding head-time curve (Eq. 21a) should be drawn, and the area under it measured, out to the time

of occurrence of zero velocity in the conduit. This area should equal the momentum of the water in the conduit under initial conditions, as indicated above, where v is the initial conduit velocity.

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CHAPTER 36

POWERHOUSE SUBSTRUCTURE

1. Introduction. The purpose of the powerhouse is to support and house the hydraulic and electrical equipment. The powerhouse is readily divided into two sections, as follows:

(*a*) The substructure to support the equipment and to provide the necessary waterways (treated in this chapter).

(*b*) The superstructure or building to house and protect the equipment from the elements (treated in Chapter 37).

The substructure may form an integral part of the dam and intake structure as in Fig. 3. In other cases the substructure may be remote from the dam, the dam, intake, and powerhouse being entirely separate structures, as at Nantahala, Fig. 15, Chapter 37, and Fig 17, Chapter 39. In low-head plants, usually 50-ft head or less, the powerhouse substructure with its intakes acts as one section of the dam.

2. General Arrangement. In modern practice the hydroelectric generating units are nearly always placed in a single row approximately normal to the direction of flow through the powerhouse. This arrangement not only avoids greater complication in the water passageways but also facilitates the handling of equipment by a traveling crane. The substructure is thus divided into a series of bays: one bay for each main unit; one for the exciter units, unless the exciters are mounted on the main units; and an entrance or working bay.

Clearances in the powerhouse between the different pieces of apparatus should be ample. There should be an unobstructed aisle not less than 8 ft in width from one end of the powerhouse to the other. It is usual to draw two imaginary lines for this aisle and to locate the edge of generators, governors, stairways, etc., on these lines. This improves the appearance of the aisle as the sides are then well defined. See Fig. 5, Chapter 37, Conowingo Interior.

Many companies have adopted as standard a minimum of 4 or 5 ft. clearance between all pieces of apparatus outside the main aisle. Exceptions, of course, are made to this rule if such space is very infrequently used by the operators.

For large installations requiring both an operating and maintenance force the switchboard may be located at any convenient place. For smaller stations where only one or two men are required for operation and maintenance the

switchboard is generally located as near as possible to the generators and at the center of maintenance activities. In large plants the switchboard is often remote from the generating equipment and is placed in its own operating room. In times of operating emergency there is a distinct advantage in this remoteness and isolation. This arrangement permits the construction of a pipe room or conduit room directly beneath the operating room, thus simplifying the electrical control installation and maintenance problems.

Governors, pumps, exciters, ventilating fans, and other small apparatus should be arranged so that a tour of inspection from the switchboard to and from all points requiring attention will be as short as possible. This feature requires careful study in complicated stations.

3. Working Bay. The working or entrance bay is usually at one end of the building and provides space where the equipment may be unloaded and in which it may be placed when being repaired. Its size depends on the dimensions of the equipment and is usually fixed by the size of the generator. It should be large enough to permit the entrance of a truck, trailer, or railroad car and to provide ample space for unloading. If the design includes a railroad track spur across the working bay, consideration should be given to the addition of a third rail so that the gage of the transformer transfer truck may be accommodated. Pillow blocks, either raised or flush, assembly plates, and floor openings for shaft accommodation should be incorporated into the floor slab with adequate column support for the erection of such equipment as the water wheels and generator rotor. It is often prudent to design the working bay as large as 1½ times the operating bay. During the erection of the units, this space is usually filled to its utmost capacity with miscellaneous apparatus which is being assembled previous to placement in its final position. In the event of an emergency breakdown, it may be necessary to repair an entire unit very rapidly. A unit out of service may be of greater value to its system in terms of its capacity than its energy, hence its early return to service is paramount.

If the entrance track or highway is at a higher elevation than the generator floor, the working bay may be elevated and the equipment lowered to the main floor by traveling crane. In many cases, however, this may mean that the elevation of the crane rail must be at a correspondingly higher elevation, thus increasing the height and the cost of the superstructure. Therefore, whenever possible, it is desirable to have the working bay at the same elevation as, or a lower elevation than, the generator floor. In designing the working bay it is necessary for the designer to figure out in advance just where each part for a unit could be placed under the crane when it is unloaded from the railroad car or truck or when a unit is stripped for complete overhaul. In general, the working bay is merely large enough to accommodate all the equipment for one unit at a single time. Usually the space occupied by the working bay is about equal to one operating bay.

4. Provision for Transformers. The subject of transformers is covered in Sections 30 to 42, inclusive, of Chapter 41, and we are interested here only

in designing the powerhouse so that the transformers may be conveniently arranged and handled.

The main transformers through which the voltage is stepped up from generator voltage to transmission voltage were formerly placed in a separate building or else in the powerhouse. Present-day practice is to place them either outdoors or inside the powerhouse, but, in any event, quite close to the generators so as to obtain leads as short as practicable.

Withdrawing the cores of the main transformers, which is sometimes necessary for repair, often requires more head room than any other operation. Consequently, the transformers are usually mounted on trucks (as at Norwood, Fig. 12, Chapter 37). Nearby is either a transfer truck, by means of which they may be moved to a position under the main traveling crane, or a secondary crane which may pick up the transformers bodily, carry them to proper position, and lower them into a pit in the working bay or to another spot low enough so that the core may be successfully withdrawn. The transformer core withdrawal location should afford complete protection from the weather, including reasonable warmth (to prevent sweating) and freedom from moisture and precipitation; also reasonable access to the bulk-oil handling plant for tank filling, draining, and re-circulation; and, finally, warm-air dryout facilities. If these operations are carried out on a concrete floor a protective covering of tarpaulins or a liberal sawdust cover is desirable.

5. Arrangement of Exciters. The electrical problem of excitation of the generators is discussed in Sections 24 to 29, inclusive, of Chapter 41. In many large plants one or more house units, consisting of a turbine and a direct-current generator, are provided so that the plant may be started up without dependence on any outside source of power and the main generator may be excited from the house unit or units. This usually requires an extra bay or bays in the substructure. In many cases, the extra cost of this provision is not justified, especially when plants are thoroughly interconnected so that an outside source of power may be depended upon. The extra expense of the additional bay or bays is frequently eliminated by utilizing direct-connected exciter units on the same shaft as each main generator or by having motor-generator sets which may utilize current from an outside source.

For the operation of auxiliary services, gates, lights, pumps, etc., and for the starting of excitation under emergency conditions when all transmission lines are cold, a gasoline-engine-operated generator set is sometimes installed as a stand-by. An important advantage of this and of motor-generator sets is that they take up very little room and require no additional bays in the substructure. A source of power that does not rely on transmission lines is essential for the operation of spillway gates and pumps in an emergency.

6. Types of Substructure. Powerhouse substructures are built exclusively of concrete and reinforced concrete. The general types of substructure may be classified according to the type of turbine setting adopted. Powerhouse substructures are designed to meet each particular situation but may be classified as follows:

(a) For open flumes.
(b) For vertical concrete spiral casings.
(c) For vertical metal spiral casings.
(d) For horizontal metal spiral casings.
(e) For impulse turbines.

The various types of casings are described in Chapter 38, and the conditions governing the choice of type are stated in the same chapter. The dimensions of the open flume and spiral casings are generally fixed by the turbine manufacturers, although changes may be made by them to conform to the necessities and desires of the engineer. At the lower heads, the decision between the use of the propeller type or the Francis type should be made early in the study of design. The design and dimensions of water passages having been decided upon, general drawings of the water passages are supplied by the turbine manufacturer.

7. Open-flume Settings. The Sewall's Island installation shown in Fig. 1 is a typical open-flume setting. The head gates may be inside or outside the substructure. At Sewall's Island they are inside the substructure and handled by the powerhouse crane. The support for the generator, spanning the open flume, should be designed for the weight of the generator, the shaft, the runner, and the hydraulic thrust. Ample allowance should be made for impact and vibration, as surges may cause rapid fluctuations in the hydraulic thrust. The maximum allowable deflection of the support is also fixed by the clearances in the turbine runner. The details of the method of support should have the approval of the manufacturers of the hydraulic turbine and of the generator. In settings of this type the generator must be set above flood-water elevation.

Attention is called here to the necessity for every possible simplification in the design and construction of such low-head small-capacity plants as the above. In such plants annual operating cost is usually a large percentage of the total operating cost, and some type of automatic or semiautomatic operation is often called for, as more specifically described in Sections 14, 15, and 16 of Chapter 39. In open-flume settings the mean inlet velocity generally does not average over 7½% of the spouting velocity (see Section 16, Chapter 39).

8. Vertical Concrete Spiral Setting. As indicated in Fig. 27, Chapter 38, concrete scrolls are applicable for capacities from a few thousand kilowatts up to capacities of about 80,000 kw, and for heads from 20 ft to about 90 ft. However, the use of this type of setting for higher capacities and heads above 60 ft is usually questionable because of the complications in reinforced-concrete design involved. In accordance with Fig. 28 and Section 19 of Chapter 38, the normal mean velocity in concrete scrolls should not exceed about 14% of the spouting velocity ($\sqrt{2gh}$). If proper attention is given to streamlining at the intake (Chapter 27) and in the scroll, the velocities herein given may sometimes be exceeded without unduly increasing losses. Velocities as high as 60% over normal have been used and

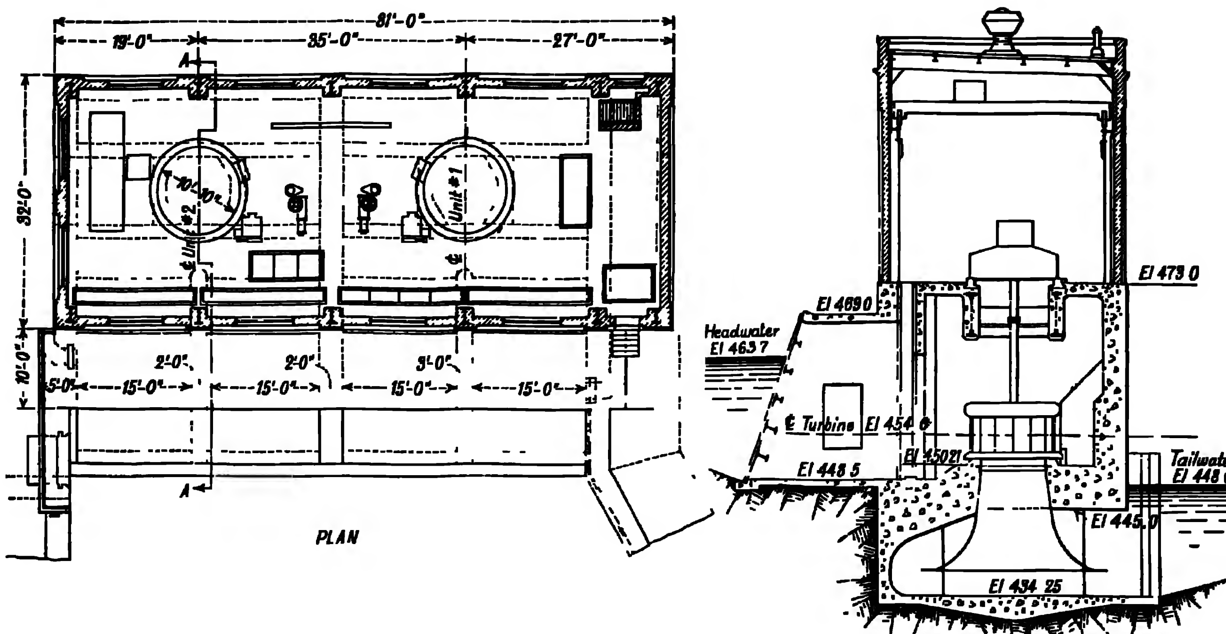
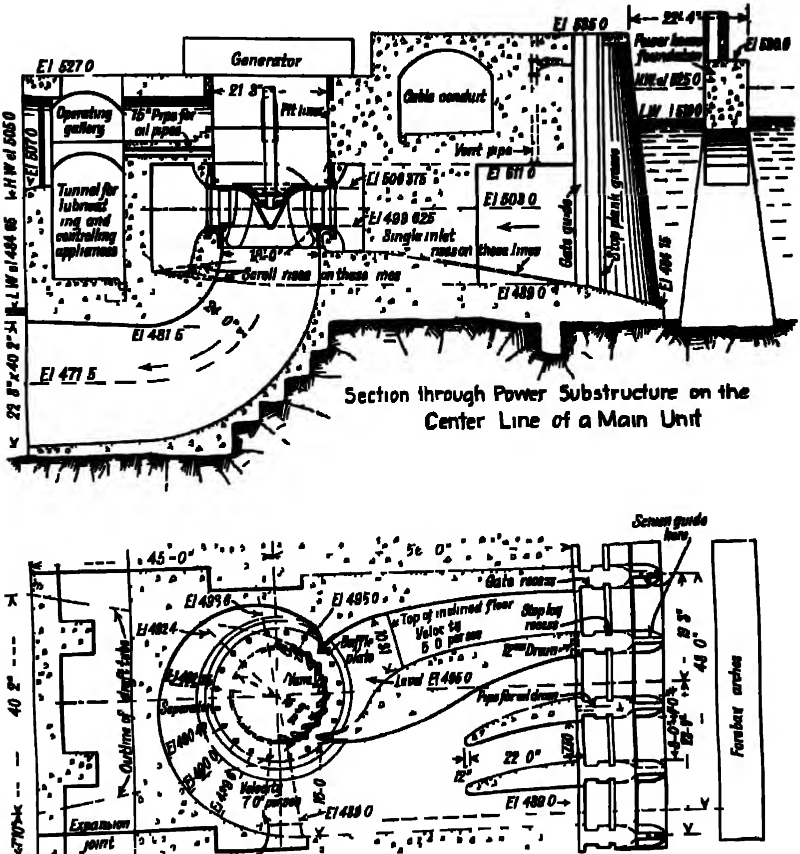


Fig. 1 Sewall's Island development, Black River, Watertown, N. Y., Central New York Power Corp. Two units at 1250 hp each. Runner propeller type, runner diameter 100 in., 150 rpm, head 15 5 ft, unit discharge 865 cu ft per second.

have resulted in a decrease of efficiency of about 1% [6]. However, abnormal velocities should not be used in passageways of the substructure without careful study and confirmatory tests. The high-speed Kaplan runner (see



Plan of Intakes and Draft Tube for a Main Unit.

FIG. 2. Powerhouse substructure, Keokuk development, Mississippi River. Mississippi River Power Co. Fifteen units at 12,000 hp each. Runner Francis type, runner diameter 139 in., 57.7 rpm, head 34 ft, unit discharge 3500 cu ft per second.

Section 2, Chapter 38) has greatly increased the number of plants for which this type of substructure becomes economical.

The large discharge per unit (13,000 sec-ft at Bonneville, Ore.) has materially increased the size of water passages and made the problem of the structural designs of the substructure more difficult. This problem has been met by including piers, islands, splitters, and stay vanes in the scroll case

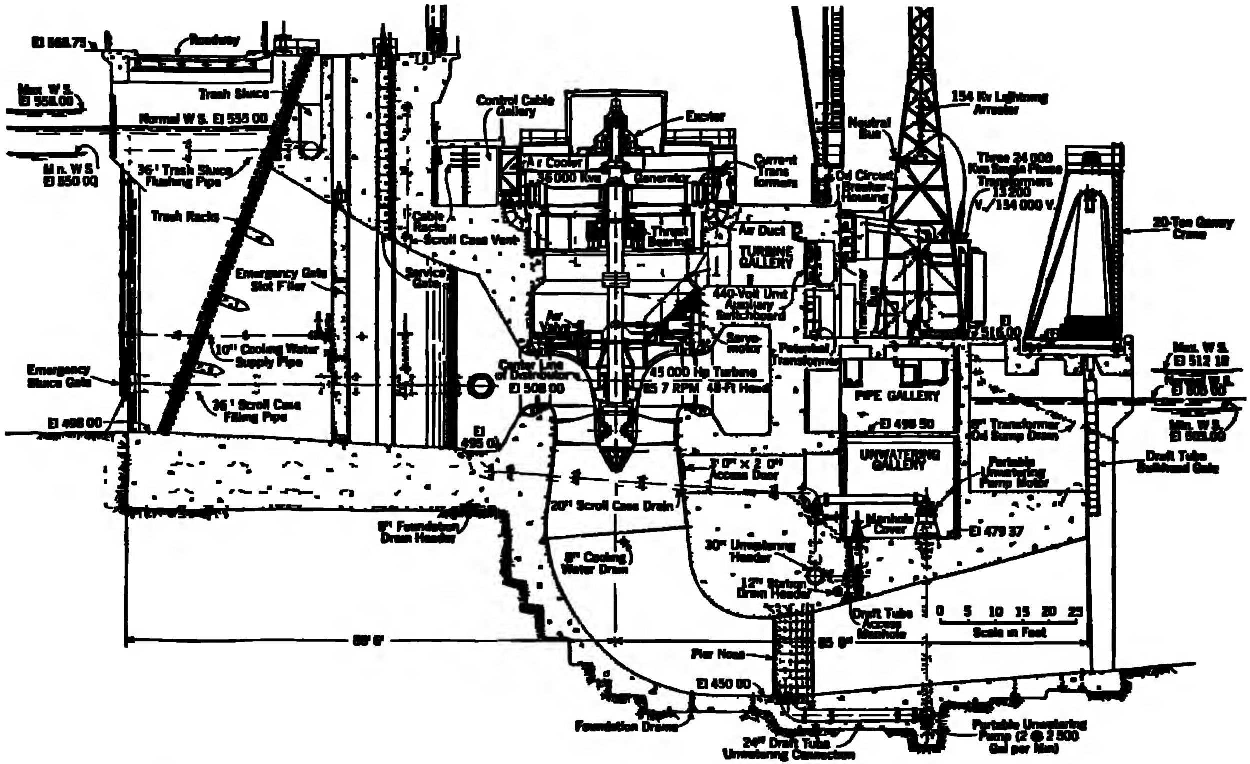


FIG 3 Cross-section through Wheeler power plant Tennessee River Tennessee Valley Authority Four units at 45-000 hp each Runner fixed-propeller runner diameter 264 in 85 7 rpm head 48 ft, unit discharge 9200 cu ft per second (*Trans A S C E* Vol 106 Fig 7 p 340 1941)

and draft tube to carry the weight above and/or the tension in the scroll case. In planning such piers and islands designers have given considerable attention to streamlining so that the over-all efficiency will not decrease because of the presence of the obstruction. When such piers and islands are designed with care, it is possible actually to increase efficiency over what

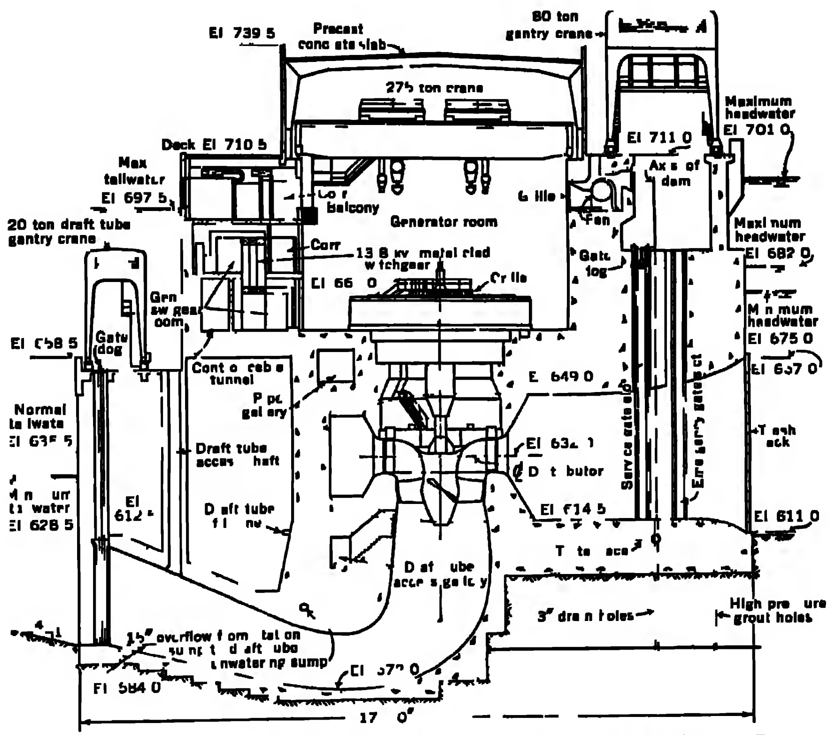


FIG. 4. Cross-section through Chickamauga power plant, Tennessee River, Tennessee Valley Authority. Three units at 36 000 hp each. Runner, Kaplan runner diameter 264 in.; 75 rpm; head 36 ft. Unit discharge 10 700 cu ft per second.

it would have been without the pier, were that omission structurally practicable.

In this type of setting the weight of the generator, the shaft, the runner, and the hydraulic thrust is carried through the speed ring of the turbine to the concrete around the draft tube.

The concrete spiral casing is in reality a pipe, and considerable reinforcement may have to be provided to take the upward pressure on the top of the casing. This upward pressure is tied to the base of the spiral casing where it is balanced by the downward pressure. It is permissible to figure on the weight of the concrete in balancing part of the upward pressure. The

generator load must not be considered, as it may not be in place when the casing is under full head. Water hammer (see Chapter 34) should be included in the upward pressure.

The upward pressure should be carried to the base both through the walls of the casing and through the speed ring of the turbine This should be

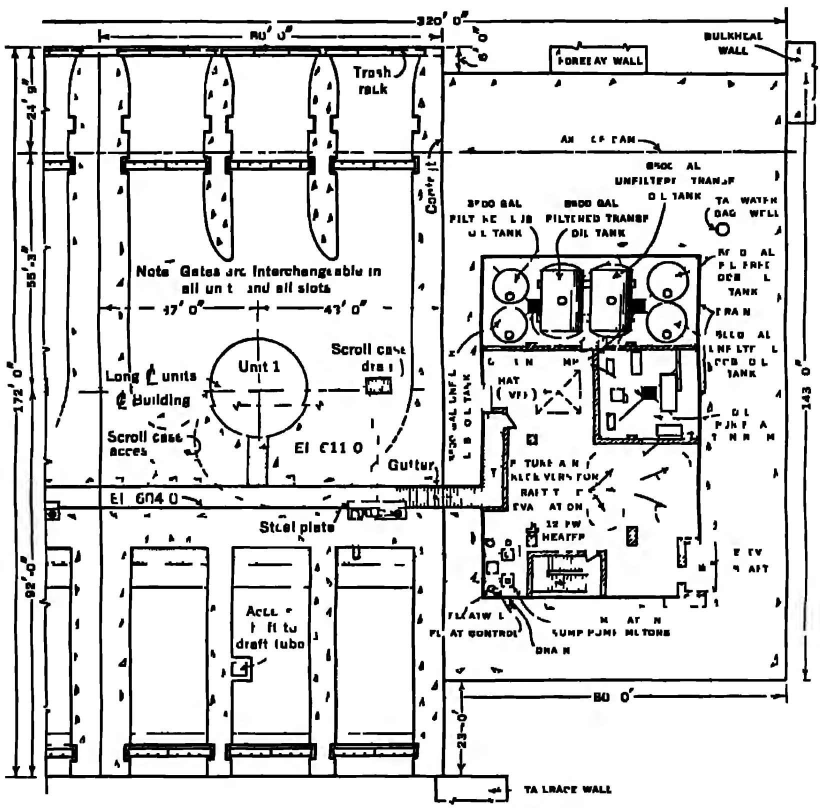


FIG 5 Horizontal section through Chickamauga power plant Tennessee River Tennessee Valley Authority Plan of Unit 1 shown Units 2 and 3 similar (See also Fig 4)

insisted upon, because the slab against which the upward pressure will occur cannot span the casing walls without upward deflection, and such deflection will be resisted by the concrete above and below the speed ring of the turbine and may cause cracking unless the concrete is properly reinforced A positive tie should therefore be provided through the speed ring of the turbine Very wide spiral casings may have partition walls, as in Fig 2, to reduce the span of the top of the casing and the width of the intake gates In Fig 2, attention is called to the rather elaborate streamlining of the piers and islands

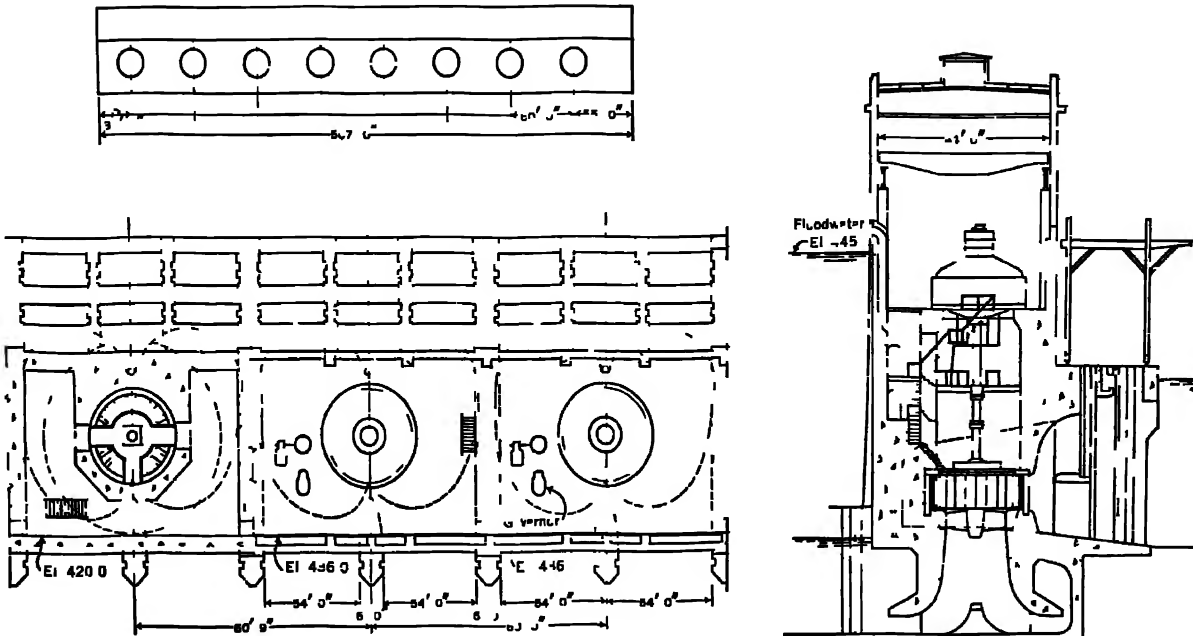


FIG 6 Ohio Falls Ohio River Louisville Ky Louisville Gas and Electric Co Eight units at 15 000 hp each Runner fixed-propeller, runner diameter 180 in 100 rpm head 37 ft unit discharge 4500 cu ft per second (Report of Hydraulic Power Committee, National Electric Light Association Publication 256-26, Fig. 11, p 17)

in the scroll. It should be noted that such streamlining is much less elaborate in some later designs, such as those shown in Figs. 4, 5, and 6. See also Section 9 as to the advisability of such streamlining. Figure 3 is a cross-section through the Wheeler concrete-cased plant. Safe Harbor, Fig. 33, Chapter 38, is another example of a plant of the same general type.

Figure 4 is a cross-section through the Chickamauga power plant, and Fig. 5 a horizontal section through the same plant. The high discharge per unit, 10,500 cu ft per second at 36-ft head, is notable, as are the large spans in the concrete scroll case. The simplicity of the streamlining in the scroll case should also be noted in comparison with the greater complexity of curvature of water passage used in the much earlier Keokuk plant, shown in Fig. 2.

9. Streamlining of Approach Passageways. From the standpoint of hydraulic efficiency, it is desirable to take the water into the substructure at

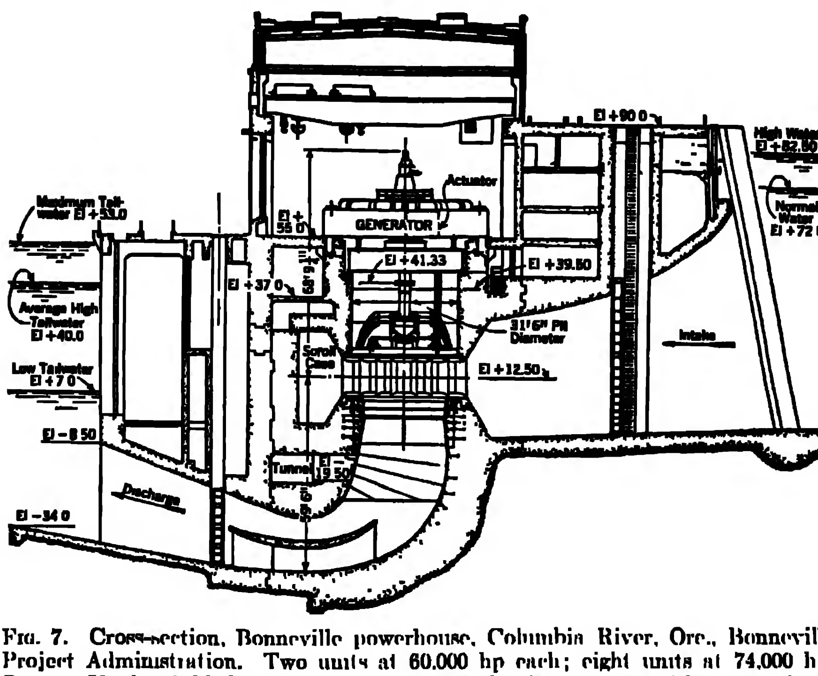


FIG. 7. Cross-section, Bonneville powerhouse, Columbia River, Ore., Bonneville Project Administration. Two units at 60,000 hp each; eight units at 74,000 hp Runner Kaplan 5 blades, runner diameter 280 in., 75 rpm, 60,000-hp units, head 50 ft, unit discharge 13,000 cu ft per second; 74,000-hp units, head 60 ft, unit discharge 14,000 cu ft per second. (*Trans. A.S.C.E.*, Vol. 106, p. 384, Fig. 23, 1941.)

as low a velocity as practicable, and to accelerate it gradually through the scroll case to the speed ring, and then, when it has passed the turbine, to decelerate it gradually to the tailrace.

In Fig. 7 is shown the cross-section of the Bonneville units, and in Fig. 8 the plan of the Bonneville scroll case substantially as constructed. The

rather elaborate streamlining is reminiscent of that used in the earlier Keokuk plant shown in Fig 2 Model tests indicate that a simple design of scroll is superior to that used in the Bonneville powerhouse It will be noted

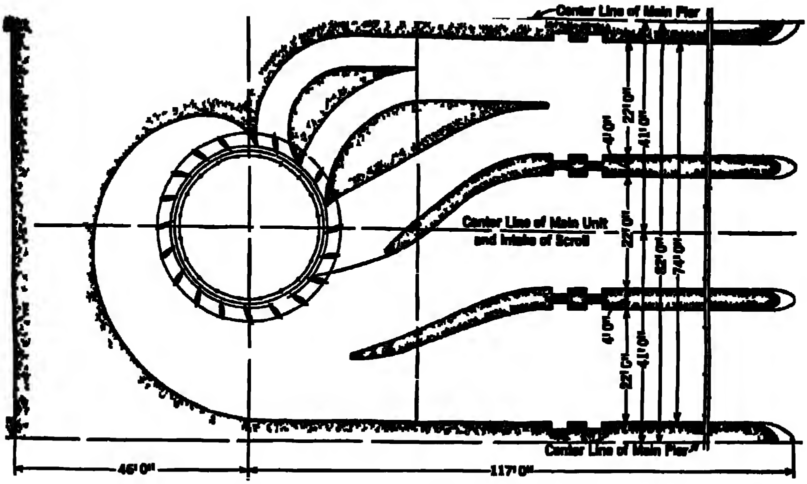


Fig 8 Plan of scroll case Bonneville powerhouse Columbia River Bonneville Project Administration (Trans ASCE, Vol 106 p 384 Fig 24 1941)

(Fig 8) that there is a multiplicity of piers and islands, the purpose of which is to guide the water into the turbine without the formation of eddies It would appear that the additional skin friction introduced by the extra pier

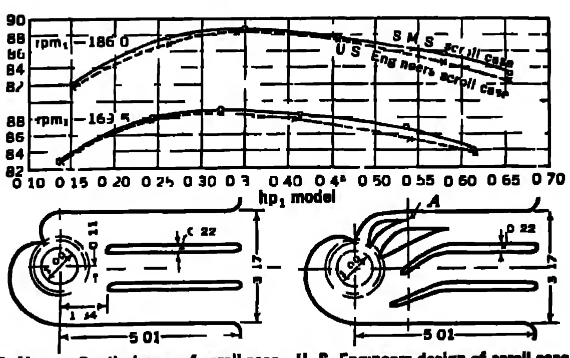


Fig 9 Comparison of Bonneville model scroll case tests Bonneville powerhouse Columbia River Oregon Bonneville Project Administration

area more than compensated for any reduction of eddy loss Figure 9 shows the comparison of model tests on the U S Engineer Office's design of scroll and a 16-in model runner homologous to the Bonneville turbine These

comparisons, made at model speeds corresponding to 50-ft and 60-ft heads, show that the simple design of scroll is superior to the more complicated one that was used. It is understood that structural considerations made it necessary to use the U. S. Engineer's design. Where it is not necessary to carry the intermediate piers too close to the speed ring of the turbine, the simpler design is preferable.*

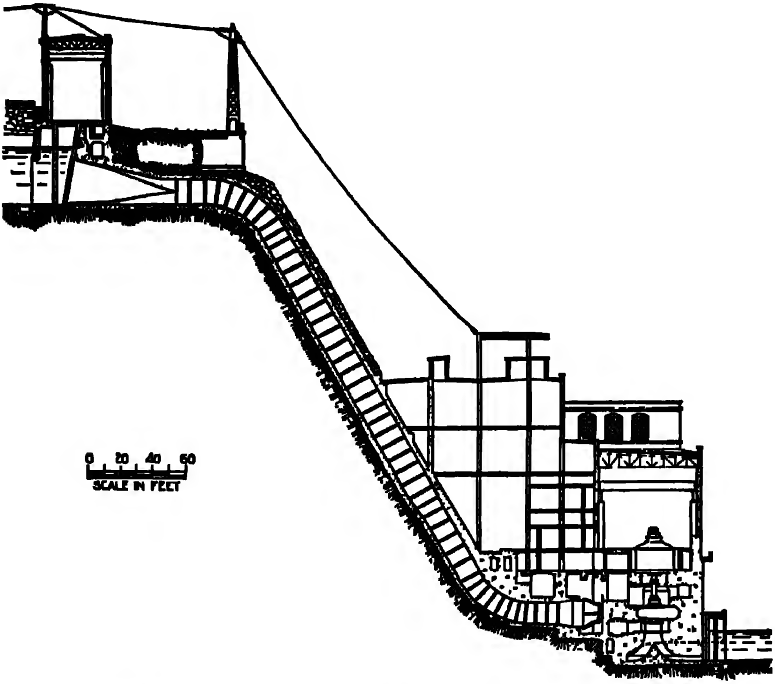
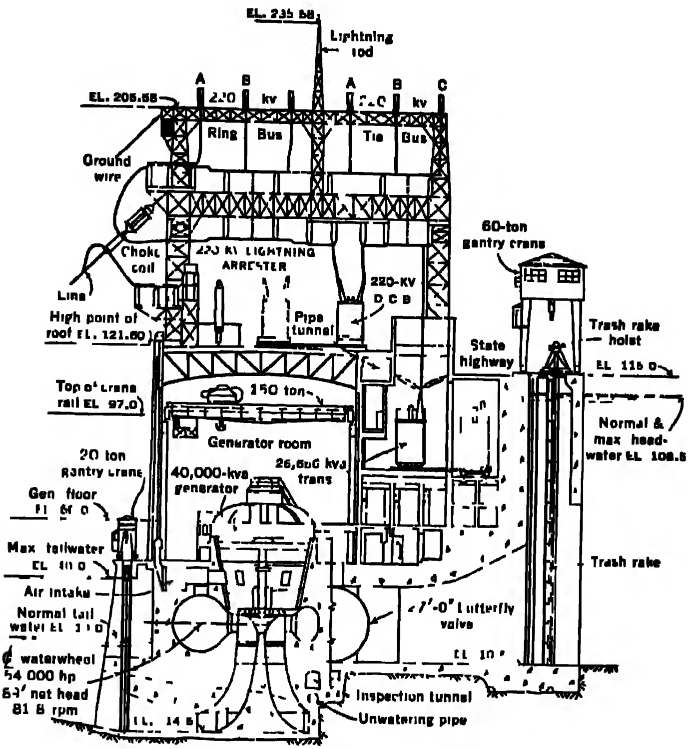


FIG. 10. Cross-section of the Queenston-Chippewa development, the Hydroelectric Power Commission of Ontario. Nine units, five 55,000-hp and four 58,000-hp. Runner Francis, runner diameters 112.5 in. and 113.1 in., 187.5 rpm, head 305 ft, unit discharge 2200 cu ft per second. (*Trans. A S C E.*, Vol. 95, p. 11, Fig. 10, 1941.)

Structural considerations would rarely require the design of such elaborate streamlining as that indicated in the Bonneville scroll. As a rule, complicated formwork materially increases costs and unless it materially increases efficiency there is no reason for adopting it. For most conditions, Fig. 5 represents the more usual and desirable practice in regard to scroll case piers. For information on the design of the intake for vertical concrete spiral settings, the reader is referred to Chapter 27, "Intakes." Figures 31, 32, and 33 of Chapter 38 show other examples of the use of concrete spiral-cased units in substructure.

* See p. 383, Ref. 6, bibliography of this chapter.

10. Metal Spiral-cased Settings for Vertical Units. As indicated by Fig. 27 of Chapter 38, metal spiral-cased settings are applicable for any capacity and for heads above 40 or 50 ft. Even at the lower heads the question of the economics of a steel spiral scroll versus a reinforced-concrete



Cross-section
Conowingo station

FIG. 11 Cross-section of Conowingo development, Susquehanna River, Maryland Philadelphia Electric Co. system. Seven units at 54,000 hp each. Runner Francis, runner diameter 194 in., 81.8 rpm. head 89 ft Average unit discharge 6300 cu ft per second.

spiral scroll should be very thoroughly examined. In many cases steel spiral scrolls prove more economical than concrete scrolls at these low heads because of the high cost of the complicated formwork required with low heads. In passing, it may be mentioned that the authors have found the designers prone to underestimate the cost of such formwork in the preliminary estimates on which the choice of setting is often based.

In accordance with Fig. 28 and Section 19 of Chapter 38, the mean velocity in steel spiral casings should not exceed 10% of the spouting velocity

($v = \sqrt{2gh}$), but in any event should not exceed about 20 ft per second. In smooth cast-iron or cast-steel scrolls, a somewhat higher velocity is sometimes used up to 20% of spouting velocity.

In the design of metal-cased spiral scroll cases, no dependence is placed on the restraining power of the surrounding concrete to resist the internal stresses of the scroll, and the metal scroll is designed to take care of all internal stresses. Actual unit stresses are usually low, as some allowance (generally at least ⅛ in.) must be made for wear of the scroll.

Figure 10 is a cross-section showing the Queenston-Chippewa Development of the Hydroelectric Power Commission of Ontario, Canada, and Figs.

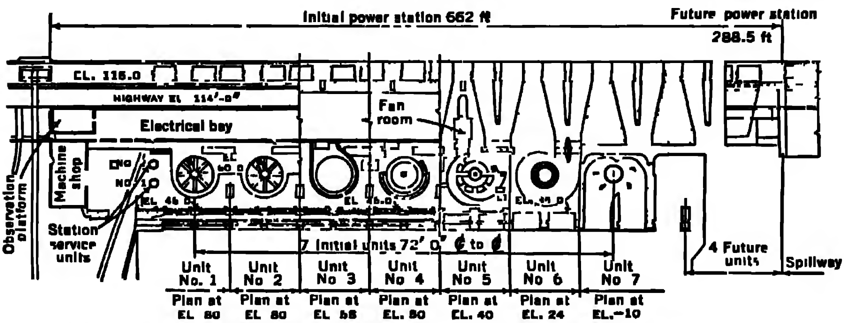


FIG. 12. Plan of substructure, Conowingo development, Susquehanna River, Maryland, Philadelphia Electric Co. system.

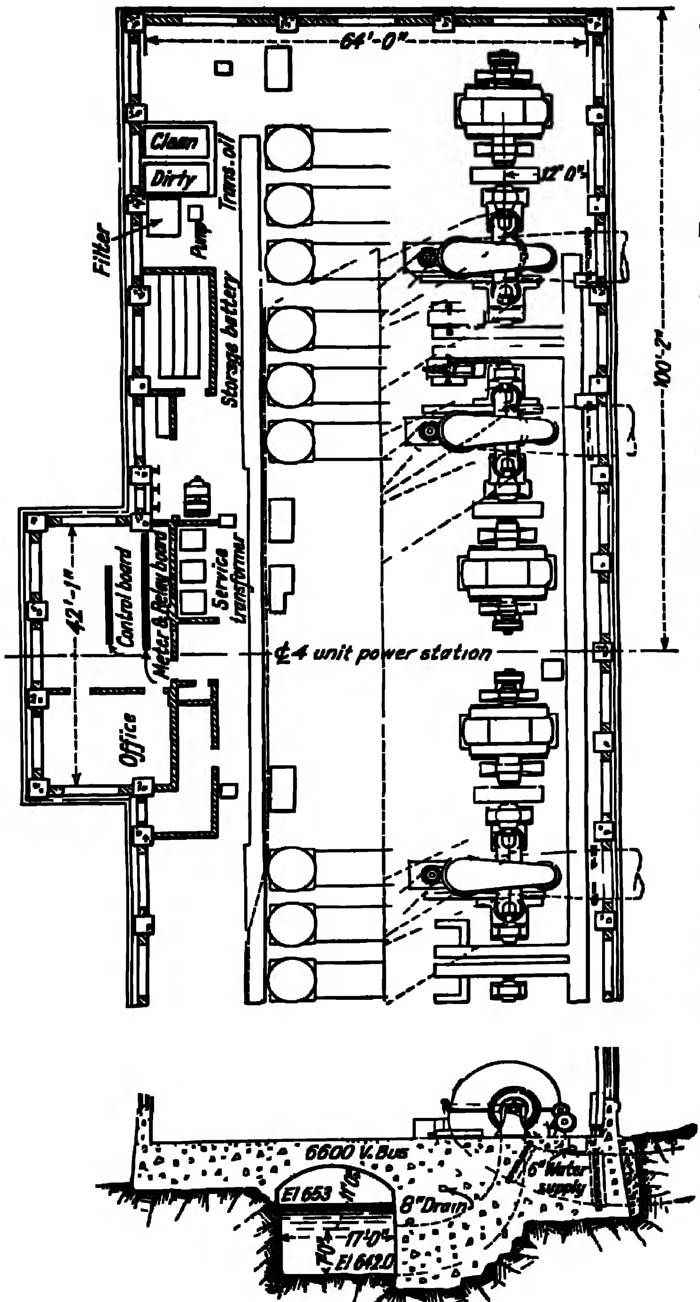
11 and 12 show the Conowingo power plant of a subsidiary of Philadelphia Electric Company. Figure 11 is a cross-section of the plant, and Fig 12 a plan showing horizontal sections of the substructure at various elevations.

Both the Queenston-Chippewa and the Conowingo development use metal-cased spiral settings in their substructure. Additional samples of metal-cased spiral settings are shown in Figs. 26, 34, 38, and 47 of Chapter 38 and Fig. 17 of Chapter 39.

Many scroll cases are now welded instead of riveted as was formerly the universal custom. See Chapter 31, "Steel Pipe."

11. Metal Spiral-cased Settings for Horizontal Units. Horizontal metal spiral-cased settings were formerly quite popular at all heads, but in modern practice they are confined to high heads, and, even there, the vertical metal spiral case setting is replacing them, as indicated by Pit River No. 5 plant, Fig. 47, Chapter 38; and Nantahala, Fig. 17, Chapter 39.

Figure 13 shows an example of typical substructure for horizontal units with metal spiral casings. By lowering the penstock to provide a vertical riser pipe between the penstocks and the spiral casing, the units may be revolved about the vertical riser and made to occupy any desired position. The arrangement in which the axis of the unit is at right angles to the center line of the penstock, as shown, is the usual one, as it provides for a higher



F: Plan of Bennetts Bridge development, Salmon River, New York, Buffalo Niagara Electric Corp. Four units at 10,000 hp each, runner double horizontal Francis with flywheel head 245 ft, unit discharge 380 cu ft per second.

penstock and usually at less cost. Another example of a substructure containing horizontal metal spiral casing is shown in Figs. 39 and 40 of Chapter 38.

12. Substructure for Impulse Wheels. Figures 14 and 15 show typical substructures for impulse turbines. The details are somewhat less complicated than those for other types of turbines and are fixed mainly by the requirements of turbine and generator builders. No special features of design can be discussed in this instance, as the types of installations vary greatly according to requirements. Other settings for impulse wheels are shown in Figs. 44 and 46 of Chapter 38.

13. Draft Tubes. Draft-tube types, described in Section 20, Chapter 38, are usually fixed by the recommendations of the turbine manufacturer, who furnishes detailed dimensions. The manufacturer is usually willing to conform, within certain limits, to the desires of the engineer, in order to meet special requirements of the substructure design, although it is necessary to follow exactly the final dimensions furnished by the manufacturer in order that his guarantees may not be invalidated.

Most of the advances in draft-tube design have been made by the various turbine manufacturers. However, it must be remembered that it is the owner and not the turbine manufacturer who chiefly benefits from any increase in draft-tube efficiency. Accordingly, it is believed that, for large-capacity installations, it is often worth while for the owner to go to the additional expense of making model tests under the particular conditions of the proposed installation in order to determine the most economical and efficient design. Many tests of various types of draft tubes have been made [6, 12].

The horizontal splitter in the draft tube sometimes used in large installations has been found to give a considerable increase in efficiency, as demonstrated by the Bonneville draft tube shown in Fig. 7.* Vertical splitters in draft tubes are also useful in transmitting the load to the foundation.

In some cases flood waters which without the splitter would pass over the spillway have been admitted to the lower part of the draft tube and have acted as an accelerator on the discharge from the unit, creating a somewhat greater effective head on the unit than it would otherwise have had.

14. Pipe Galleries and Electrical Conduits. The space in front of the turbines and just below the main generator floor, as in Figs. 4, 6, and 7, is generally occupied by a continuous gallery from end to end of the plant. This gallery usually contains all the piping for the operation of the plant, including the piping for the governor system, lubricating oil, water, etc.

The electrical cable is often also carried in this same gallery, but is sometimes laid through a separate gallery or tunnel on a rack in which the cables are laid in troughs so that any cable desired is accessible. For convenience, it is desirable both to have runs of conduits laid in the generator floor short and to get the cables down into a gallery like the one described as soon as

* See p. 383, Ref. 6, bibliography of this chapter.

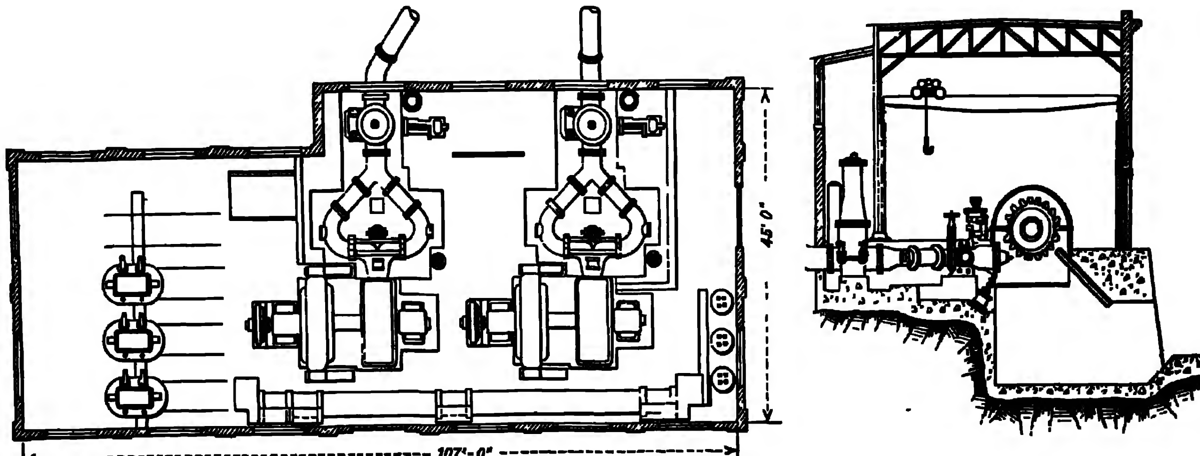


Fig 14. Plan and cross-section of Boulder Canyon development. Mid Boulder Creek. Public Service Co. of Colorado. Two units at 14,250 hp each, runner impulse, runner diameter 89.5 in at pitch line, 400 rpm, head 1830 ft static, 1637 ft effective, unit discharge 95.7 cu ft per second.

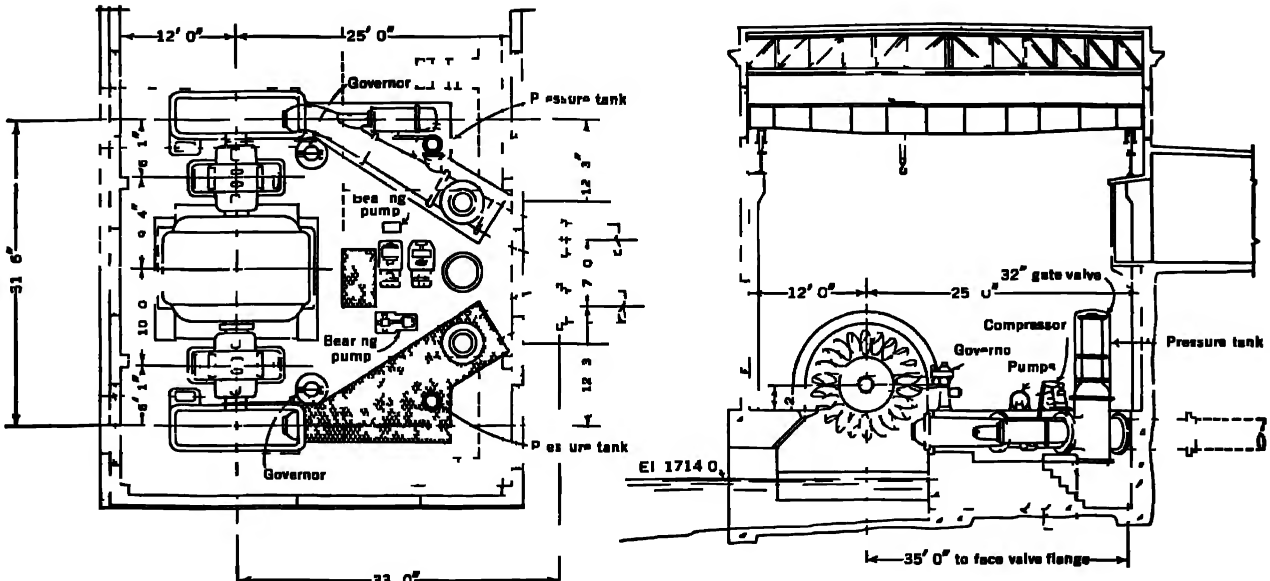


FIG 15 Balch development Kings River California Pacific Gas and Electric Co One unit of 40 000 hp runner double overhung impulse pitch diameter runner 115 in 360 rpm head 2336 ft unit discharge 720 cu ft per second Report of the Hydraulic Power Committee, National Electric Light Association Publication 256-26 Fig 10 p 16 1926)

practicable. The subject of conduits and ducts is fully covered in Section 19, Chapter 42.

15. Ventilation. The ventilation requirements of generators, discussed in Section 9 of Chapter 41, frequently need attention in the design of the substructure.

16. Remarks on Details. Certain rules for detail of design to safeguard the powerhouse operations are mandatory in some states. Copies of these rules pertaining to fire doors; details of stairs, ladders, and walkways; clearances, railings, and similar items may be obtained through the insurance companies. They contain many valuable suggestions.

Oil tanks should be placed in the basement or buried underground. They should be capable of being drained into the tailrace by the operation of a valve which would be accessible in case of fire.

Data on the generator torque should be obtained from the manufacturer. Provision should be made to prevent twisting of the generator if it is supported on a high concrete barrel, and to prevent overturning of a horizontal generator if it is not firmly anchored to solid foundation.

The substructure should always be made entirely fireproof. The provision is frequently an economy in view of resultant lower fire insurance rates. Small doors and windows may be of wood, provided they are not adjacent to apparatus which is likely to take fire.

All small pieces of apparatus should be placed on concrete pedestals about 6 in. high, both to improve their appearance and to prevent possible damage by water on the floor.

17. Substructure Floors. It is general practice to stop the concreting of the substructure a foot or so short of the finished level of the main floor (generator floor of the powerhouse) to facilitate the placing of the many conduits or ducts always required here. After all the conduits and ducts called for by the plans have been placed, concreting may be completed to the finished grade of the floor. Usually the concreting is not completed until after all heavy equipment has been placed in final position so as to avoid injury to the floor. For completing this floor as well as any other, it is important that stiff concrete should be used, as wet concrete often results in dusty and unsound floors.

A terrazzo finish for the main floor gives an excellent, hard, permanent surface. The difficulty with this type of floor, however, is that it must be placed while work goes forward on generators and other equipment, and that the necessary grinding required to lay it produces an objectionably large amount of dust.

It is worth while, merely from the standpoint of presenting a good appearance, to spend a little extra time and money on the generator floor. Clay tile, rubber tile, and linoleum tile all have been successfully used and all look well. The tiling is frequently carried up the inside walls above head height, adding materially to the appearance.

For small or inaccessible plants where cost must be kept to the minimum for basement and gallery floors and galleries, expensive floor finishes are seldom used. Utility is the main consideration. The floors must be as dust free as possible, for concrete dust is an enemy of all revolving machinery. Various proprietary hardeners have been used successfully.

It is entirely practicable to produce an almost dust-free floor by the following method: Stop the concreting 1½ in. short of final grade. Clean off thoroughly, and roughen the old concrete surface. With the surface entirely clean and moist, place a top coat consisting of 1 part Portland cement and 2 parts clean quartz sand, placed as dry as one would place mortar, in a test briquette mold in the laboratory; screed and ram this very stiff mortar into place, and finish without any wetting or any dusting with dry Portland cement. No hardening compound is necessary in such a floor. Even painting with a suitable concrete floor paint to keep the floors from dusting will not be necessary for many years if the work is properly done. On floors placed of ordinary concrete and with ordinary workmanship, frequent painting is necessary to prevent them from dusting.

18. Provisions for Dewatering Scroll and Draft Tubes. Usually scroll cases are above tailwater elevation and may be dewatered by drains of ample size. Sometimes, especially in small plants, no special provision for unwatering the draft tubes is made, the designer assuming that, when and if the operator desires to unwater the draft tube, he will find some way of placing a pump behind the tailrace piers. However, in all large-capacity substructures and, the authors believe, in most small ones, special provision should be made for entirely dewatering the draft tubes without too much inconvenience.

There are many ways of accomplishing this. One way is to have each draft tube drain to a valve on a tee, which connects to a pump-header line designed to take water from the bottom of the draft tube. The header connects to duplicate pumps which take the water and raise it to discharge above the tailrace elevation. Vertical-shaft centrifugal pumps with the motor situated above tailwater level are suitable for this service. In determining the capacity of the pumps, allowance for very heavy leakage past or through tailrace stoplogs or gates should be assumed because it frequently turns out that such leakage is far greater than the designer anticipated, necessitating in some cases costly delays and even the installation of temporary supplementary pumps each time the draft tubes are dewatered. A compressed-air connection to the draft tube should always be provided where the turbine runner is below tailwater elevation. The compressed air, in such cases, may be used for depressing the water level in the draft tube so that the unit may be used as a synchronous condenser without the runner encountering resistance from the churning up of the water.

19. Sump in Substructure. Every substructure should contain a sump to which all seepage and leakage drains. From this sump the water is pumped and discharged above tailwater elevation. Vertical-shaft centrifugal

pumps with the motor located well above the highest water level are suitable and should be provided in duplicate at least. Sometimes there should be two pumps, each capable of taking maximum estimated seepage, plus two other pumps of much larger capacity designed to take the water out of the plant within 24 hr if the plant gets flooded.

Expensive accidents have occurred that could have been avoided had the designer adopted such simple and relatively inexpensive expedients as the above.

20. Tailrace Piers, Gates, and Stoplogs. In order to permit the unwatering of the draft tubes, tailrace gates or stoplogs are necessary at the exit of the draft tubes. Tailrace piers are generally necessary to support such gates and stoplogs, and some means of operating them must generally be provided. Such gates and stoplogs are similar to those provided at intakes, and the reader is referred to Chapter 27 on intakes for a discussion of their design. There is one important difference, however; whereas it is necessary to operate intake gates very frequently, tailrace gates or tailrace stoplogs are operated only when the draft tube is unwatered for repairs or inspection purposes. As this seldom happens more than once a year, it follows that it is economical to use equipment which is much less expensive and which requires a much longer time to place.

It is necessary to provide tailrace piers with slots for containing the gates and stoplogs With large-capacity units, there are usually several of these tailrace piers per unit. They generally project into the draft tube quite a way and perform the additional function of helping to support the load of the structure and shortening the span that would otherwise be necessary. Tailrace piers are shown in Figs. 2, 3, 4, 5, 6, and 7 of this chapter.

If the span between tailrace piers does not exceed about 16 ft, creosoted wooden stoplogs are entirely suitable, as pointed out in Section 30, Chapter 27. Wooden stoplogs generally have to be jacked into place, and this can be greatly facilitated by providing anchorage in the piers as illustrated in Figs. 65 and 66 of Chapter 27.

In a plant with large-capacity units, the spans between piers are often too great for timber stoplogs, and, accordingly, steel stoplogs or sectionalized steel gates are used. Only enough stoplogs are provided for closing of one draft tube at a time.

With large units, as in Figs. 4, 5, 7, and 12, it is necessary to deck over the tailrace piers and to provide a track with a light crane to handle the steel stoplogs or gates and also such tools and material as may be required in dewatered draft tubes.

21. Dimensions and Volume of Substructure. For preliminary design and estimating purposes, the dimensions and volume of any proposed substructure may be approximately determined in the following manner. Obtain the diameter and elevation of the units in relation to tailwater by the methods outlined in Sections 9 to 16, inclusive, Chapter 38. Section 20 in

Chapter 38 discusses the form and dimensions of the runners. Figures 15, 18, 29, 30, and 42 of Chapter 38 are the principal ones used for determining substructure dimensions. A working bay of about the same size as the space required for one unit must be added, and, if house units are to be used, additional space must be provided for excitation. Then, having the approximate over-all dimensions of the power plant substructure, determine the gross volume. Multiply this gross volume figure by 0.6 and thus determine the approximate number of cubic yards of concrete that will be re-

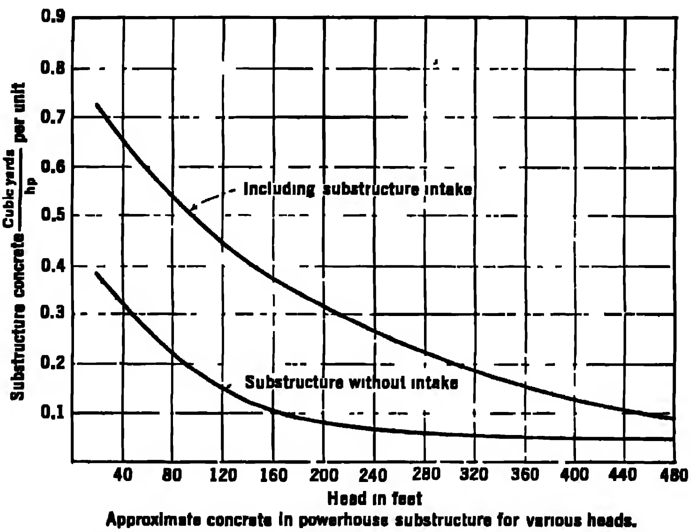


FIG. 16. Approximate concrete in powerhouse substructure for various heads.

quired for the powerhouse substructure. This method is rough, and precise enough only for the first preliminary estimate. Another method is indicated in Section 22.

22. Quantities for Substructure. In addition to the above method, one may estimate for preliminary purposes the amount of concrete in the proposed power plant from the actual content of existing plants of similar unit capacity, head, and speed. The amount of concrete in a powerhouse substructure varies greatly from one plant to another, so that it is most difficult to apply an adequate empirical rule for preliminary estimation of quantities.

As an aid to the estimator, concrete quantities for a large number of typical plants have been given in Table 1. Here are listed, in order of increasing heads, concrete quantities for plants with propeller-type units, Francis-type units, and impulse-type units.

The quantities shown in Table 1 are plotted in Fig. 16, using the ratio of cu yd/hp per unit as the index of volume of concrete. This is the most

adequate method of expressing a definite relationship. The two main considerations are given as (*a*) substructure including intake, (*b*) substructure without intake. The two curves represented are based on Table 1.

Example of the use of Fig. 16 is shown in Table 1, pages 778–781.

Assume that the gross head is 80 ft, that the size of units is to be 5000 hp, and that there will be four units. In the course of making a preliminary estimate, it is desired to know approximately how much concrete there will be in the substructure of the proposed powerhouse, assuming that substructure will include intake.

Using Fig. 16, it is found that 80-ft head corresponds to an ordinate value of 0.54 on the substructure including intake curve. This is a purely mathematical ordinate equal to the cubic yards divided by the horsepower. Therefore, in our case:

$$\frac{\text{cu yd}}{5000} = 0.54 \quad \text{from which} \quad \text{cu yd} = 2700$$

or each 5000-hp unit will require 2700 cu yd of concrete in its substructure. As there are to be four units, and the erection bay will occupy about the space of one unit, the approximate total concrete required for the substructure (including intake) of such a plant is $5 \times 2700 = 13{,}500$ cu yd. The results obtained are rough and uncertain, being applicable only to the preliminary estimate.

At the expenditure of much more time, an experienced engineer can, by utilizing the method given in Section 21 and by giving consideration to foundation conditions as well as to tailwater conditions, make a more certain estimate of excavation and concrete required for a proposed powerhouse substructure.

NOTES (for Table 1)

1. Redevelopment.
2. Intake excavation = 4230 cu yd
3. Addition to existing powerhouse
4. Installed in existing building
5. Building space includes all habitable space within substructure plus the gross volume of the superstructure. Plant designed and constructed by Engineering and Construction Department of Tennessee Valley Authority.
6. Plant designed and constructed by the Safe Harbor Water Power Corporation. Intakes for 12 units = 70,596 cu yd Substructure for 7 units = 68,691 cu yd Total = 139,287 cu yd Ultimate 12 units = 510,000.
7. Semi-outdoor installation
8. Includes turbine foundation.
9. Intake excavation = 358,740 cu yd
10. Plant designed and constructed by the U. S. Bureau of Reclamation.
11. Plant designed and constructed by Stone & Webster Engineering Corporation
12. Data supplied by the Philadelphia Electric Company, designed and constructed by Stone & Webster Engineering Corporation.
13. Plant designed by J. G. White Engineering Corporation
14. Originally designed by the J. G. White Engineering Corporation, plants now incorporated in the Tennessee Valley Authority system
15. Designed by Chas. T. Main, Inc.

CONCRETE, STRUCTURAL, AND PHYSICAL QUANTI

Reference Number	Plant Name	State	Year Built	Head on Plant, ft	Total Hp of Plant, thousands	Number of Units	Hp Each Turbine, thousands	Kva Each Generator
1	Lake Doris	Nebr.	1930	10	0.3	1	0.3	0.25
2	Siphon Drop	Ariz.	1925	13	2.2	2	1.1	1.0
3	Tomahawk	Wisc.	1937	15.8	3.8	2	1.9	1.7
4	Taftsville	Vt.	1943	20	0.675	1	0.675	0.625
5	Kelleys Falls	N. H.	1925	21	1.5	1	1.5	1.250
6	Enosburg Falls	Vt.	1944	21	0.890	1	0.89	0.75
7	Merced Falls	Calif.	1930	27	4.2	1	4.2	4.0
8	Millers Falls	Mass.	1938	30-32	1.5	1	1.5	1.25
9	Roanoke Rapids	N. C.	1924	30	3.2	1	3.2	3.0
10	Chippewa Falls	Wisc.	1928	20.6	30.0	6	5.0	4.5
11	Garvins Falls	N. H.	1925	30	8.2	2	3.5 4.7	3.0 4.0
12	Rock Island	Wash.	1933	32	84.0	4	21.0	16.667
13	Guntersville	Ala.	1939	36	102.0	3	34.0	27.0
14	Chickamauga	Tenn.	1940	36	108.0	3	36.0	30.0
15	Ohio Falls	Ky.	1927	37	120.0	8	15.0	12.55
16	Clarks Falls	Vt.	1937	41	4.0	1	4.0	3.5
17	Pickwick	Tenn.	1938	43	192.0	4	48.0	40
18	Park Station	Va.	1940	40	3.88	1	3.88	3.0
19	Kentucky	Ky.	1944	48	220	5	44.0	35.0
20	Wheeler	Ala.	1936	48	180.0	4	45.0	36
21	Hardwick	Vt.	1937	50	1.05	1	1.050	1.0
22	Black Eagle	Mont.	1927	50	27.9	3	9.3	7.0
23	Watts Bar	Tenn.	1942	52	210	5	42.0	33.3
24	Safe Harbor	Pa.	1931	55	297.5	7	42.5	60 cu yd 36.0 25 cu yd 37.50
25	Fort Loudon	Tenn.	1943	65	88	2	44.0	35.6
26	Erickson	Nebr.	1930	19	0.51	1	0.51	0.438
27	Iron Mountain	Mich.	1923–1924	30	9.6	3	3.2	3.0
28	Red Bridge	Mass.	1933	49	3.0	1	3.0	2.25
29	Middlesex	Vt.	1928	50	4.8	2	2.4	2.0
30	Black Canyon	Ida.	1924	77	12.0	2	6.0	5.0
31	Parker	Ariz.-Calif.	1939	77	160.0	4	40.0	30.0
32	Conowingo	Md.	1928	89	378.0	7	54.0	40.0
33	Pilot Butte	Wyo.	1929	90	2.4	2	1.2	1.0
34	Osage	Mo.	1931	90	180.0	6	30.0	22.888
35	Grandfather Falls	Wisc.	1938	91	23.0	2	14.7 8.35	13.75 7.85
36	Milton	Vt.	1929	95	4.5	1	4.5	3.75
37	Douglas	Tenn.	1943	100	83.0	2	41.5	33.3
38	Cherokee	Tenn.	1942	100	83.0	2	41.5	33.3
39	Bartletts Ferry	Ga.	1926	112	44.0	2	22.0	18.75

Notes on p. 777.

TIES OF VARIOUS TYPICAL HYDRO PLANTS

Type of Turbine	Turbine Rpm	Unit Discharge, cu ft per second	Concrete in Substructure, thousands cu yd	Is Intake Incl. in Substructure or is it a Separate Structure?	Thousands Cu Ft Space in Superstructure	Thousands Cu Yd Excavation for Powerhouse	Thousands Cu Yd Excavation Tailrace	Notes
Propeller	164	300	0 05	Separate	4.0			1, 11
Propeller	112 5	887	2 40	Included	58.0	9 645	2.772	2, 10
Propeller	120	1,300	2 70	Included	60.0	2.3	Included in p h.	
Kaplan	257	37[illegible]	0 292	Separate	6.6			3, 15
Propeller	180	720	0 73	Separate	18 0	0.24		11
Kaplan	225	452	0 27	Included		0 442	3 350	4, 15
Propeller	128.6	2 010	1.5	Included	None	2.3		
Kaplan	277	460	0 435	Separate	17 9	0.70	0.1	15
Propeller	164	1,050	1 85	Separate	30 0	4.8	Included in p h	11
Propeller	138 5	1 750	15 0	Included	459 0	16.0	43 0	
Propeller	150	1,160	2 55	Included	60 0	1 3		1, 11
Propeller	138 5	1 570						
Hand adj. prop.	100	7 000	114 0	Included	380 0	110.0	76 0	11
Kaplan	69 2	9 800	84 1	Included	2,850 0	194 22	Included in p h.	5
Kaplan	75	10 700	142 0	Included	2,570 0	161 152	Included in p h	5
Propeller	100	4 500	59 0	Included	1,478.0	150 0	Included in p h	
Kaplan	240	1 020	1 46	Separate	38.3	1 32	4 5	15
Kaplan	81 8	11,200	160 2	Separate	3,070.0	1 688 0	Included in p.h	5
Motor adj prop	257	850	1 13	Separate	Outdoor generator	2.8	3 1	11
Kaplan	78 3	9,000	187 0	Included	1,745.0	499 468	Included in p.h	5
Fixed prop	85 7	9,200	134 0	Separate	1,750.0	256 317	Included in p h	5
Propeller	514	216	0 158	Separate		0 11	0 209	4, 15
Propeller	180	1,880	8 22	Included	351.9	6 0	47.19	15
Kaplan	94 7	8 300	102 1	Included	1,675.0	95 233	101 897	5
	60 cu yd	8 800	151 3	Included	4 314 0	142 100	182.431	6
Kaplan	109 1 25 cu yd 100							
Kaplan	105.8	6 700	99 7	Included	928.0	92 149	22 618	5
Francis	180	270	0.10	Separate	15.0	1 0	4.400	11
Francis	120	1,070	5 9	Included	330.0	3 9	1.800	11
Francis	150	675	0.6	Included		0 40		4, 15
Francis	200	485	1.217	Separate	40 4	0 853	1 929	15
Francis	225	800	0 214	Separate	60 0	3 268	.	10
Francis	94 7	5,250	50.0	Separate	800 00	71.405	. ...	10
Francis	81 8	6,320	194 3	Included	5,894 0	115 6	193.20	12
Francis	514	146	0.427	Separate	19 0	14 80	Prob. incl tailrace and forebay excav.	10
Francis	112 5	3 300	170.0	Included	800.0	66 0	31.0	7, 11
Francis	180	1 650	2 1	Separate	90.0	2.3	Included in p.h.	
	200	940						
Francis	277	495	0 708	Separate	35 5	0.37	3.33	15
Francis	94.7	4 400	53.6	Separate	900 0	32.734	26.68	5
Francis	94 7	4 400	59.3	Separate	900.0	37.056	21.897	5
Francis	150	2 000	6.8	Separate	326 0	10.0	10.0	11

CONCRETE, STRUCTURAL, AND PHYSICAL QUANTI

Reference Number	Plant Name	State	Year Built	Head on Plant, ft	Total Hp of Plant, thousands	Number of Units	Hp Each Turbine, thousands	Kva Each Generator
40	Swinging Bridge No. 1	N. Y.	1928	124	6.9	1	6.9	6.25
41	Swinging Bridge No. 2	N. Y.	1938	124	9.5	1	9.5	7.5
42	Elephant Butte	N. M.	1938	132	34.5	3	11.5	9.0
43	Newport	Vt.	1936	135	4.8	2	2.4	1.875
44	Rock Island	Tenn.	1924	142	22.2	1	22.2	18.75
45	Blue Ridge	Ga.	1928	165	26.8	1	26.8	25.0
46	Norris	Tenn.	1936	165	132.0	2	66.0	56.0
47	Bullard's Bar	Calif.	1924	166	10.0	1	10 0	8.1
48	Seminoe	Wyo.	1936	168	45.0	3	15.0	12.0
49	Rio Development	N. Y.	1927	170	14.0	2	7.0	6.25
50	Saluda	S. C.	1930	180	220.0	4	55.0	40.625
51	Green Mountain	Colo.	1939	185	30.0	2	15.0	12.000
52	Hiwassee	N. C.	1940	190	80.0	1	80.0	64.0
53	Calera	Ecuador, S. A.	1936	196	1.2	2	0.6	0.563
54	Hat Creek No. 2	Calif.	1921	198	15.0	1	15.0	12.5
55	Baker River	Wash.	1927	216	48.8	2	24.4	19.5
56	Hat Creek No. 1	Calif.	1921	217	15.0	1	15 0	12.5
57	Melones	Calif.	1927	230	35.8	2	17.9	13.5
58	Narrows	Calif.	1942	240	13.5	1	13.5	11.0
59	Salt Springs	Calif.	1931	244	13 5	1	13.5	11 0
60	Kern Canyon	Calif.	1921	262	12.0	1	12.0	10.6
61	Ocoee No. 3	Tenn.	1943	280	33.5	1	33.5	30.0
62	Pit No. 3	Calif.	1925	315	99.0	3	33.0	27.0
63	Spaulding No. 3	Calif.	1929	318	8.0	1	8.0	7.0
64	Fontana	N. C.	1944	330	183.0	2	91.5	75.0
65	Grand Coulee, Left Powerhouse	Wash.	1941	325	900.0	6	150.0	108.0
66	Shasta	Calif.	1944	330	206 0	2	103.0	75.0
67	Spaulding No. 2	Calif.	1928	344	5.3	1	5.3	4.2
68	Mollys Falls	Vt.	1926	350	7.0	1	7.0	6.25
69	Apalachia	Tenn.	1943	360	106.0	2	53.0	40.0
70	Huango	Ecuador, S. A.	1939	368	2.1	3	0.7	0.625
71	Pit No. 1	Calif.	1922	454	80.0	2	40.0	35.0
72	Hoover	Ariz.-Nev.	1936	475	1,380.0	12	115.0	82.5
				465	55.0	1	55.0	40.0
73	Prospect No. 2	Ore.	1928	590	46.8	2	23.4	20.0
74	Pit No. 5	Calif.	1944	630	200.0	4	50.0	40.0
75	Dutch Flat	Calif.	1943	643	29.0	1	29 0	27.5
76	Prospect No. 3	Ore.	1932	693	10.0	1	10.0	9.0
77	Pinnacles Dev.	Va.	1938	700	13.5	3	4.5	3.75
78	Caribou	Calif.	1921	1,149	90.0	3	30.0	22.22
79	Phoenix	Calif.	1940	1,187	2 3	1	2.3	2.0
80	Tiger Creek	Calif.	1931	1,218	72.0	2	36.0	30.0
81	Spring Gap	Calif.	1921	1,365	9.5	1	9.5	7.5
82	El Dorado	Calif.	1924	1,910	28.0	2	14.0	12.5
83	Balch	Calif.	1927	2,336	40.0	1	40.0	33.0
84	Bucks Creek	Calif.	1928	2,562	70.0	2	35.0	25.0

Notes on p. 777.

Continued

TIES OF VARIOUS TYPICAL HYDRO PLANTS

Type of Turbine	Turbine Rpm	Unit Discharge, cu ft per second	Concrete in Substructure, thousands cu yd	Is Intake Incl. in Substructure or is it a Separate Structure?	Thousands Cu Ft Space in Superstructure	Thousands Cu Yd Excavation for Powerhouse	Thousands Cu Yd Excavation Tailrace	Notes
Francis	300	570	0.805	Separate	42.8	1.0	0.8	15
Francis	240	1,014	0.9	Separate	47.5	1.2	1.5	15
Francis	257	890	5.572	Included	177.0	20.865	. ...	10
Francis	360	187	0.600	Separate	33.4	0.68	1.8	15
Francis	164	1,700	....	Separate	. ....	....	. .....	14
Francis	....	......	....	......	. . ....	....	. ...	14
Francis	112.5	4,300	22.4	Separate	1,155.0	106.377	Included in p.h.	5
Francis	257	650	0.4	Separate	50.0	1.4	0.9	
Francis	225	900	13.041	Included	315.0	36.048	.. ..	10
Francis	360	880	1.475	Separate	70.1	1.8	7.5	15
Francis	138.5	3,200		Separate	736.0	......	.. . ..	
Francis	257	815	4.5	Separate	105.0	66.0	13.794	10
Francis	120	4,300	9.2	Separate	390.0	16.682	18.497	5
Francis Horiz.	1,200	33		Separate	....	.	. ...	13
Francis	225	000		Separate	124.7		. . ..	
Francis	300	1,100	3.1	Separate	1,318.0	56.0	4.7	11
Francis	225	600	1.400	Separate	131.500	. .... .	... .	8
Francis	277	810	0.900	Separate	342.0	2.3	....	
Francis	300	790	1.57	Separate	102.0	5.0	Included in p.h.	8
Francis	300	685	1.07	Separate	210.0	18.95	0.6	
Francis	257	720	0.31	Separate	150.0	4.3	.. .. .	8
Francis	200	1,200	5.4	Separate	358.0	5.2	24.948	5
Francis	225	1,130	2.2	Separate	573.0	10.2	29.4	8
Francis	450	270	0.056	Separate	124.0	......	. ....	
Francis	150	2,880	29.4	Separate	1,506.0	16.648	243.4	5
Francis	120	4,615	204.4	Included	3,700	2.000.0	1,500.0	10
Francis	138.5	3,200	58.0	Separate	1,700	394.6	.... .	10
Francis	300	185	0.1	Separate	42.5	0.215	.. .. .	8
Francis	600	210	0.45	Separate	32.9	0.65	None	15
Francis	225	1,500	9.9	Separate	571.0	9.636	0.822	5
Francis Horiz.	1.200	20	0.83	Separate	...	1.7	. .	13
Francis	257	960	1.05	Separate	465 0	14.2	111.1	8
Francis	180	2,530	199.90	Separate	4,000.0	402.2	....	9, 10
Francis	257	1,150						
Francis	514	400	4.4	Separate	144.0	6.0	2.2	
Francis	300	875	14.5	Separate	900.0	96.0	Included in p.h.	8
Francis	400	475	3.6	Separate	233.0	50.5	Included in p.h.	8
Francis	750	150	0.65	Separate	39.0	0.70	No tailrace	
Impulse	450	. ....	0.462	Separate	86.4	1.854	....	15
Impulse	171.4	350	4.7	Separate	900.0	11.5	50.0	8
Impulse	514	24	0.1	Separate	36.0	. ...	. .. .	
Impulse	225	350	1.27	Separate	455.0	4.79	13.5	
Impulse	514	65	0.10	Separate	95.0	1.2	0.150	
Impulse	300	92	.. ..	Separate	194.0	...	..... .	
Impulse	360	230	.	Separate	220.0	... ...	....	
Impulse	450	185	. .	Separate	160.0	.	. . ... .	

23. Tailrace. The tailrace is a much-neglected feature of the hydroelectric development. Engineers spend a great deal of time, money, and effort in obtaining a highly efficient intake, scroll case, and draft tube, but when it comes to the tailrace, it is often dismissed with an assumption as to the elevation of tailwater under operating conditions made on the basis of entirely inadequate data. The frequent result is that head is lost which might have been obtained at very low cost if an adequate investigation and study had been made in advance of construction.

At any site where a plant is being considered, a careful study of tailrace conditions should be undertaken. The first step is to have records showing the discharge at different elevations of tailwater. Usually a number of downstream gages should be established as they may show the presence of projecting ridges which influence tailwater elevation at the proposed plant. On many sites, as where the proposed tailrace occupies a portion of the river bed, model studies may be necessary in order to determine just where and how much excavation is economically desirable for the tailrace. Sometimes a dividing wall between tailrace and river will materially increase operating head. Every tailrace is a special problem in itself, and for many projects hydraulic model studies should be made and should form the basis for design

For a hydroelectric development already in service, where the difference in tailwater elevation with plant shut down and at full plant discharge is extreme, the tailrace conditions should be investigated, as the possible gain in head by tailrace improvement is sometimes economically advisable.

Sometimes an uneconomical amount of work is done in streamlining the tailrace. For instance, the discharge from the draft tube is often at an elevation far below the bottom of the river Sometimes, in an effort to streamline the tailrace and obtain minimum operating elevation of tailwater, the designer has transposed the tailrace from the draft tube exit up to the natural river bottom in a very gradual slope, the upgrade being in some cases as low as 1 vertical on 20 horizontal, at a cost of many thousands of cubic yards of rock excavation. Practically nothing is gained by such a procedure, and there is generally no sense in making this upgrade flatter than 1 vertical on 6 horizontal. Model tests have verified this conclusion (see also Section 20, Chapter 38).

24. Degrading Tailrace. When the river contains a large amount of alluvial or glacial deposits of appreciable depth, the construction of a dam and power plant tends to degrade the river below the plant for a long distance or to the next control point downstream. That is, the operating elevation of tailwater tends to become lower and lower throughout the years.

This degradation is due principally to the fact that the structures shut off the supply of deposits from the river downstream from the dam, but the river continues to erode. Sometimes the degrading has gone so far that the draft tubes are unsealed, and then a tailrace dam may have to be constructed to keep the tailwater up to a point where draft tubes will stay sealed. A proper investigation and study in advance of construction will generally re-

veal whether the bed of the river will degrade after the construction of the dam. If it is found that it will, the units and the draft tubes should be set low enough so that the degrading will increase the operating head and thus increase the value of the plant in future years.

25. Regrading Tailrace. There are many tailraces which accumulate deposits resulting in higher operating tailwater and a decreased head on the plant. These deposits must be removed at considerable cost or the plant suffers a permanent loss of value due to decreased head. Such deposits are often caused by locating the spillway so that it discharges into the tailrace or a short distance downstream from it, depositing the material eroded near the toe of the dam in the tailrace.

Sometimes designers, in an effort to get minimum first cost, utilize a saddle spillway which discharges into the river not far below the plant. Erosion below such a saddle spillway is often extreme, and large masses of rock are frequently deposited in the tailrace therefrom. In such cases within the experience of the authors the operating head has been decreased as much as 5 ft by floods over the saddle spillways, and, although expensive excavation has been undertaken, the lost head has not been entirely regained. Much can be done to avoid such conditions by the use of proper design. It is believed that a saddle for a power dam should usually be avoided. However, if the point where the discharge from the saddle spillway reaches the river is sufficiently far downstream, the very large resulting deposit of débris and rock may not affect the elevation of powerhouse tailwater.

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CHAPTER 37

POWERHOUSE SUPERSTRUCTURE

By Joseph H. Gandolfo *

1. General Conditions. The primary functions of the superstructure of any powerhouse are to shelter and protect the machinery and operators and to provide facilities for handling the equipment.

The layout of the power-plant equipment, including turbines, generators, exciters, switchboard, oil switches, and other auxiliary machinery, and sometimes high-tension transformers, determines the general over-all dimensions of the building. The height of a fully enclosed superstructure generally depends on the clearance which the traveling crane requires to handle the largest pieces of equipment and carry them over the other machinery while in operation. If, however, the head gates are located inside the powerhouse and are lifted by the crane, they may be the controlling influence for the height of superstructure.

The following is a list of the usual equipment for which the superstructure must provide space, but not *all* powerhouses require space for *all* these items:

1. Main generating machinery.
2. Turbine governors, pumps, and tanks.
3. Motor-generator sets, exciters.
4. Compressed-air equipment.
5. Water-supply pumps for river water and filtered water, and a filtration plant if river water is a source of drinking-water supply.
6. Switchboard and low-tension switches and buses
7. High-tension transformers and switches (if in the building).
8. Storage batteries.
9. Transil-oil tanks with centrifuge, filter and pumps.
10. Telephone and radio equipment.
11. Lubricating-oil tanks with filters and pumps.
12. Elevators and hoists (if plant is large).
13. Locker rooms and wash rooms.
14. Operating and engineering offices.
15. Machine and carpenter shops.
16. Storeroom (stock room for small parts) and sufficient space for dead storage of large spare parts.
17. Fire-fighting apparatus (fixed and portable).
18. In cold climates, boiler room for building heating and ice melting.
19. Test room with facilities for oil, water, routine, and special tests.

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Space must also be provided on the main floor, at one end of the building, for handling and dismantling the machinery. This space, which should be at the entrance to the powerhouse, is ordinarily spoken of as the working bay. (See Chapter 36.)

If the substructure includes a basement, less space will be required on the main floor, as some of the less-used auxiliary equipment can be located in the basement.

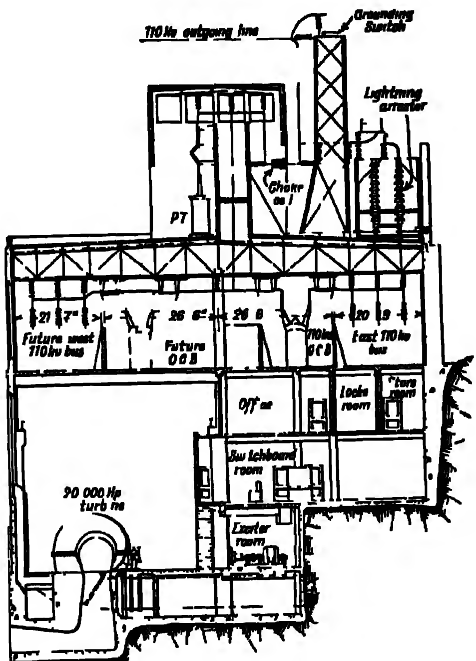


FIG. 1.—Cross-section Baker River powerhouse, Baker River, Washington. Puget Sound Power and Light Co. Two units at 24,300 hp; total capacity 48,600 hp; 230-ft head; 300 rpm; 6600 volts. (J. N. Robinson, *Electrical World*, Vol. 87, p. 548.)

It is frequently necessary to provide an ell or lean-to on the main building to obtain sufficient space for the switchboard and switches. In some large powerhouses, where the high-tension electrical equipment is inside the building, as in Fig. 1, and space is limited, the superstructure must be several stories high.

Present-day design seldom places the high-tension transformers or switching equipment within the building proper on account of the large size of such units.

2. Abbreviated Type of Superstructure. The abbreviated type of superstructure may consist of either a long building extending over the units

with the roof just high enough to clear the exciters, or a simple metal enclosure over each unit large enough to provide walking and working space inside the enclosure. In the first type the roof has large hatchways for the installation and removal of equipment, and in the latter type the entire casing is removable. Handling facilities usually consist of a gantry crane traveling the full length of the powerhouse on rails supported on the substructure. The switchboard and other necessary auxiliaries are usually placed in a structure at the shore end of the powerhouse. Working space for takedown and repairs is also provided under the gantry. Typical examples of abbreviated superstructures are indicated in Figs. 7, 12, and 13 of this chapter, Fig. 3 of Chapter 36, and Fig. 11 of Chapter 11.

At the Kentucky power plant of the Tennessee Valley Authority, located on the Tennessee River in Kentucky (Fig. 7), access to the units is by means of large hatchways in the roof of the low powerhouse superstructure. The gantry crane, of 250-ton capacity, handles the generating units, main transformers, and auxiliary equipment and operates the intake and draft-tube gates.

At the Bagnell plant of the Union Electric Light and Power Company, (Fig. 13) access to the units is also by large hatchways in the roof of the low superstructure. The gantry has only one long leg on the down-stream side carried on the substructure, and the other end runs along the top of the wall of the electrical bay. It has one short cantilever arm on the down-stream side to handle stoplogs.

In the Norwood powerhouse on the Peedee River, South Carolina (Fig. 12), each unit has its own individual casing that is removable, and each generator is equipped with its individual air-circulating system, the air being water-cooled The crane has one long leg on the downstream side. The upstream end has short stub legs running on the crest of the headwall.

Figure 16 shows an abbreviated type of powerhouse suitable for a plant in a gorge. The enclosure is just large enough to give working space around the unit, and a hatch in the roof permits access to the equipment. Handling facilities are provided by a stiff-leg derrick located on the bank above the powerhouse.

One objection to the abbreviated type of powerhouse superstructure is that, as soon as the hatch or other protective covering is removed for repairs or any other reason, the generator and auxiliaries are exposed, except for such temporary protection as may be provided by tarpaulins or other simple means. In a sudden driving snowstorm, or a heavy rain, it needs no stretch of the imagination to picture what might happen. Therefore, in northern climates where there is a great deal of snow and ice or where there are constant sudden showers, the advisability of the abbreviated type is highly questionable.

Generally, the abbreviated type is more likely to produce savings in cost at a large development with many units than at a small development of only one, two, or three units. In the former type one gantry can handle a long

line of generators and runners and the cost of a long powerhouse superstructure is thus saved. In addition, on account of the number of units, if one must be removed for repairs, it is generally not of vital importance to return it to the line in the shortest possible space of time. If weather condi-

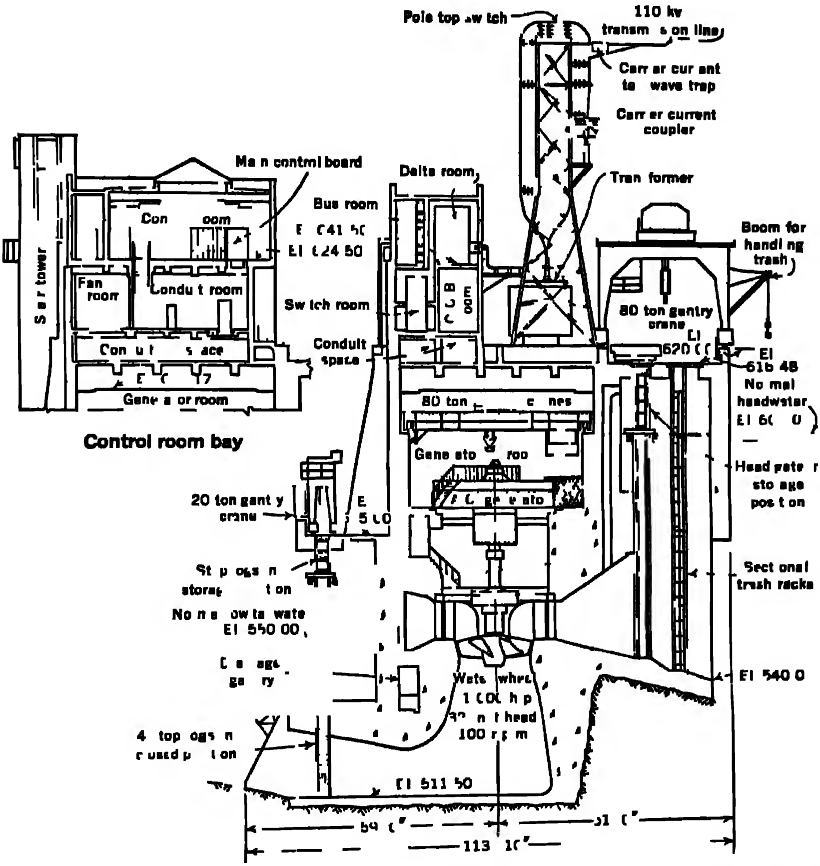


Fig 2 Cross section Rock Island powerhouse Columbia River Washington Puget Sound Power and Light Co Four units at 21 000 hp total capacity 84 000 hp 32-ft head 100 rpm

tions are unfavorable, the unit can stay idle for a day or even several days.

At a small plant, the cost of the special equipment made necessary by the abbreviated type is very likely to more than offset any savings effected in the cost of the superstructure. In any event, an enclosure must be provided for the switchboard and auxiliaries. In the abbreviated type particularly if metal enclosures over the turbines and exciters are used, artificial means of ventilation and cooling must be installed for the generators. Also,

the gantry crane is of necessity much more costly than the ordinary crane of equal capacity. However, an abbreviated superstructure may be economically justifiable even for small developments in a location where the same gantry crane can be used to service the units and to operate the head gates and draft-tube gates. (See Fig. 7.)

3. Fully Enclosed Type of Superstructure. The fully enclosed superstructure consists of a building enclosing all the apparatus and equipment with the usual exception of high-tension transformers, which are generally outdoors. In this type the crane travels on rails supported by the side walls or structural members. Typical fully enclosed superstructures are indicated in Figs. 1, 2, 3, and 11 of this chapter, Figs. 4, 7, and 11 of Chapter 36, and Figs. 7, 9, 33, and 38 of Chapter 38. Some of the details pertaining to this type of superstructure are discussed in the remainder of this chapter.

The choice between the fully enclosed superstructure and the abbreviated type is a matter that can best be determined by a careful study in each particular case. (See also Section 2.) However, many operators prefer the fully enclosed type of powerhouse where units can be dismantled and worked on without interference from the weather.

See Table 1 of Chapter 36 for quantities in various types of powerhouse construction.

4. Architectural Effect. Some power plants are located in or near centers of population, and good roads have brought many formerly remote plants within easy visiting distance of the public. If it is probable that the power plant may be visited by considerable numbers of the public, it always pays to devote attention to securing a pleasing architectural effect. Even though the plant is remote and will be seen by few people, the services of a competent architect should be utilized as it usually costs no more to obtain a pleasing appearance than an unsightly one.

Although a hydroelectric powerhouse is a purely utilitarian structure, sometimes it is considered necessary, especially in the case of a government-owned plant, not just to erect an ordinary building but to build a structure that is highly ornamental both inside and outside. In such cases, cut stone or terra-cotta trim may be used on the exterior, and tile floors, tile or glazed-brick wainscoting, or entire walls of such materials may be used on the inside. The writer has seen powerhouses in which the entire crane girder was hidden by a frieze of glazed brick.

However, there is no necessity of going into expensive designs to obtain a sturdy and handsome building, even for a government plant. Many excellent examples exist of good-looking structures of reinforced concrete, brick, and other inexpensive materials; and agreeable exteriors can be built with very little additional outlay over what a purely utilitarian structure would cost. Very attractive superstructures have been built of common brick ornamented with a small amount of precast concrete. (See Figs. 3 and 14.)

Within the building, careful consideration should be given to the use of

color as this adds much to the appearance and often nothing to the cost. The units and all their auxiliary equipment should be the same general color and should complement the wall treatment. The walls may be of glazed tile, tapestry brick, or even concrete properly painted. Such attention to detail will make a plant attractive to visitors and employees and promote good housekeeping; often it creates contentment among the staff. Few realize the eyestrain produced by clashing colors. Figure 8 shows the interior of a control room in a modern plant.

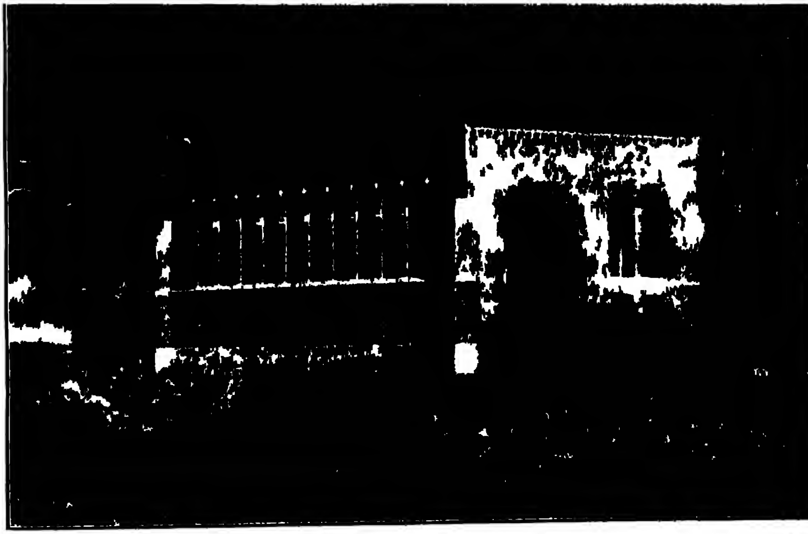


Fig. 3. Brick exterior design powerhouse. Harriman development, Deerfield River, Vermont. New England Power Co. Three units at 20 000 hp, total capacity 60 000 hp, 390-ft head, 360 rpm, 6000 volts.

In carrying out any such decorative schemes, it should be kept in mind that they should give the effect of strength, solidity, and massiveness and that this should be the controlling motif throughout. (See Fig. 5.) The light airy and fantastic have no place whatever in a powerhouse design.

5. Framework. The framework of a powerhouse superstructure should unquestionably be of structural steel. The steel design should be investigated on a unit loading basis for both a roof substation load and the usual roof loads. Such a study may well determine the most economical location for the substation. With a steel frame, when the substructure is sufficiently completed the steel for the superstructure can be quickly erected and the walls and roof completed. The early installation of the traveling crane also permits the erection of the turbine and generator while the general building construction proceeds. Steel framing in a typical powerhouse is indicated in Fig. 11, Chapter 36.

With a reinforced-concrete superstructure, the supports for the roof, crane rails, and walls are usually poured together, and considerable time elapses before erection of the turbine can proceed. Therefore, the saving in time resulting from the adoption of a structural-steel frame may considerably shorten the time for the delivery of power.

Figure 17 of Chapter 39 shows the concrete superstructure of the Nantahala powerhouse, Fig. 15 of this chapter, the exterior view of the same plant.

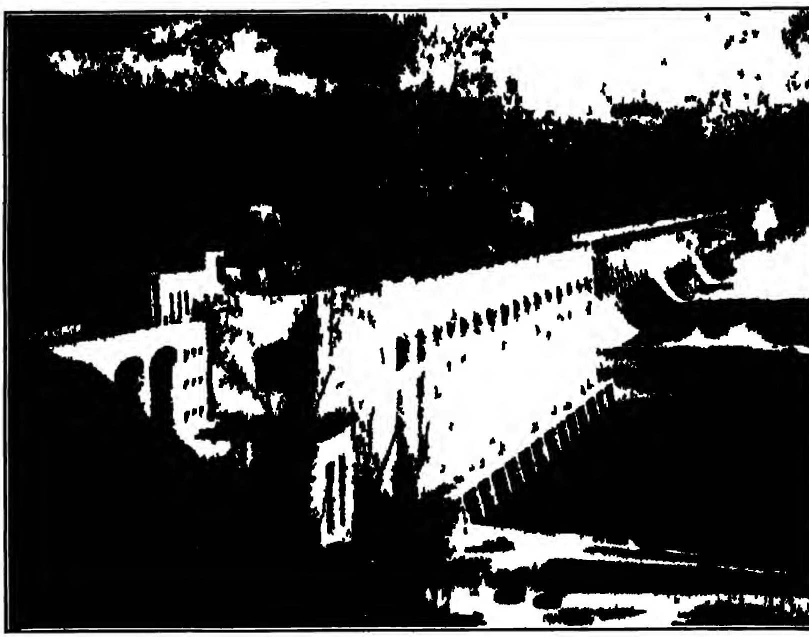


FIG. 4. Concrete exterior design powerhouse, Conowingo development, Susquehanna River, Maryland. Philadelphia Electric Co. system.

6. Walls. All powerhouses of any consequence are entirely fireproof. The following materials are suitable for walls:

1. Brick.
2. Brick veneer backed with other material.
3. Reinforced concrete.
4. Concrete blocks.
5. Stone backed with brick or other material.
6. Hollow tile (terra cotta blocks).
7. Gunite on steel lathing.

Brick of suitable color and texture gives a pleasing appearance and is probably the most widely used material in the walls of powerhouse superstructures (Figs. 3 and 14). In a structure as permanent as a powerhouse

the brickwork should always be laid up in the most approved manner. Every sixth course should be a true header course. If Flemish bond is adopted, as shown in Fig. 10, darker-colored brick can be used for the headers, thus giving a very good appearance. In the Flemish bond, all headers need not be true ones, but there must be enough true headers to secure a good bond. Generally glazed or pressed brick is used for the interior surfaces to secure good-looking and smooth walls. Plastering or painting of such surfaces is not necessary.

FIG. 5. Interior view of turbine hall, Conowingo development, Susquehanna River, Maryland. Philadelphia Electric Co. system.

Brick-veneered walls can be used, backed with concrete blocks, tile blocks, or reinforced concrete. The only advantage of such walls over solid walls of brick is that they make a good appearance on the exterior at minimum expense. Care must be taken to thoroughly anchor the brick to the backing with wall ties or true headers to prevent separation from the main wall. Such walls may be plastered or painted on the inside, if desired.

In small powerhouses the walls can be designed to support the roof with wall pilasters supporting the crane runway beams. In large powerhouses with a structural-steel frame the enclosure may be a curtain wall between floors and columns. This may be only 8 in. thick if the panel is not too large. Generally, a curtain wall not less than 12 in. thick is recommended. The advantage of the curtain-wall type of construction is that it permits

installing the crane, the roof, and some of the equipment before proceeding with the enclosing walls and thus saves valuable time in the completion schedule. The Conowingo plant (Fig. 4) is an example of curtain walls of reinforced concrete. Figure 5 shows an interior view of the same plant.

Tile and concrete block are frequently used for curtain walls, but on account of the rough surfaces of these materials they do not give as good an appearance as brick.

FIG. 6. Exterior view, Norfolk dam and powerhouse, North Fork River, Arkansas. U. S. Engineer Dept. One unit 42,000 hp, 160-ft head.

Reinforced concrete is used quite extensively for walls as its general appearance has been improved by good architectural design and new methods of construction. A flat concrete wall of any great extent without any relief never pleases the eye. Defects such as cracks or misalinement are extremely noticeable. Paneling and banding can do much to improve the appearance of a wall of this type.

One great disadvantage of reinforced concrete in powerhouse design is that this material does not readily lend itself to alterations, changes, and additions. It is of necessity monolithic.

Steel forms or steel-lined forms do not give a pleasing surface. Holes from air bubbles are practically always evident at the surface, and defects in alinement are enhanced. If wood is used for forms a pleasing effect may be secured by having all the board lines run in the same direction so that any given board line has the same elevation. Rubbing with Carborundum bricks

and patching should be done as little as possible. The effects are generally temporary, and usually nothing is gained by rubbing out the marks made by the grain of the wood.

Absorbent form lining gives a dense, weather-resistant concrete surface. It is recommended for concrete walls where a fine appearance is desired as in Fig. 9. Figure 6 is a good example of a powerhouse with concrete walls.

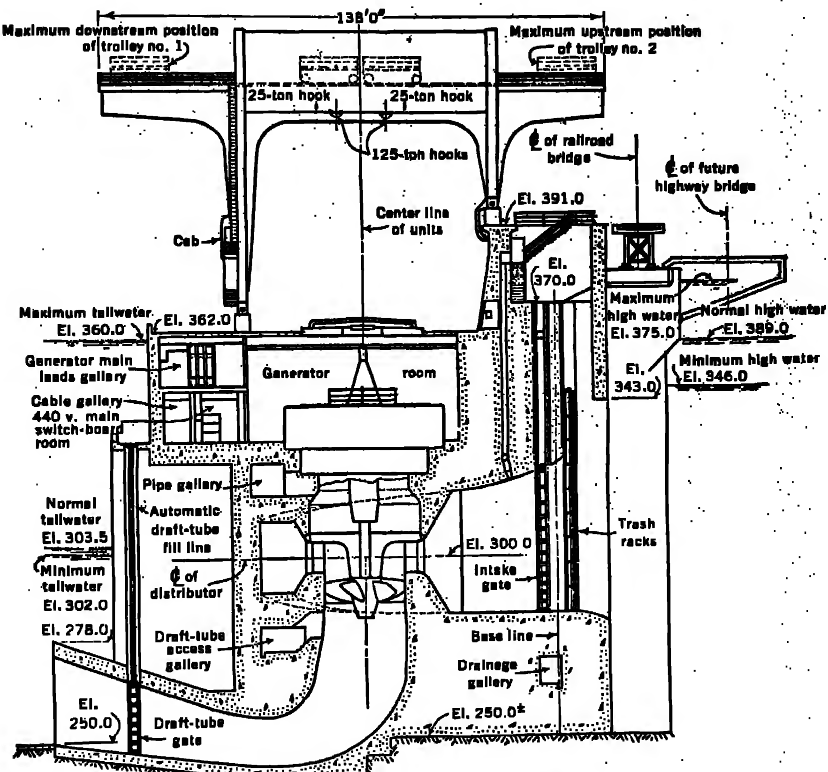


FIG. 7. Cross-section, Kentucky powerhouse, Tennessee River, Kentucky. Tennessee Valley Authority. Total capacity 180,000 hp, 48-ft head, 85.7 rpm. (E. H. Weilman, *Civil Eng.*, June 1943, p. 257.)

Stone creates a very satisfactory appearance, and where it is readily available and labor costs are not excessive it can often be used to advantage. The Schoellkopf plant at Niagara Falls is an excellent illustration of such a plant.

The choice of material for powerhouse walls is a matter of study for each project. Local conditions for various materials and labor as well as the location of the structure itself affect the selection to a large degree.

7. Doors and Windows. Sufficient ventilation is an important consideration in the design of any powerhouse, and light should always be ample wherever possible. It is necessary to have a steady and adequate flow of fresh air into the building in warm weather and to provide exits for the hot air given off by the generating equipment

It is seldom possible to have the powerhouse comfortable in extremely hot weather, even in northern latitudes, and the greatest amount of window space consistent with proper architectural effect is advisable Too great a

Fig 8. Control room Fort London powerhouse Tennessee River Tennessee Tennessee Valley Authority Two units at 44 000 hp, total capacity 88 000 hp 65-ft head, 105 rpm

window area gives the impression of weak construction, which is inconsistent with the proper treatment of power-plant structures

Much consideration is being given to glass blocks and permanently closed windows Ventilation is accomplished by means of louvers, air filters, and ductwork for distribution This system adds much to the cleanliness of the equipment and plant Mixing by dampers the filtered air with the waste heat from the generators will provide a comfortable winter temperature Controlled temperature ventilation is most important in the large switch houses where it is imperative to keep the air conditions above the dew point to avoid flashover With the normal growth of large plants, such a system lends itself to the ultimate installation of refrigeration, which may become necessary to hold the ambient temperature within the operating range

Both steel and wood sash are used in powerhouses Steel sash should be chosen wherever possible, as it reduces the fire risk to the minimum and in most localities costs very little more than wood sash

In building steel sash into the walls, if the walls are of brick, concrete blocks, or terra-cotta blocks, the sash should be set up before the walls are built, in the same manner that wood frames are erected, and the walls then built around them. The writer is of the opinion that this method should also be followed for reinforced-concrete walls.

Aluminum, copper-bearing, or stainless-steel alloys are being used for the sash and doors of many high-class structures, as they do not rust and their length of life is almost unlimited.

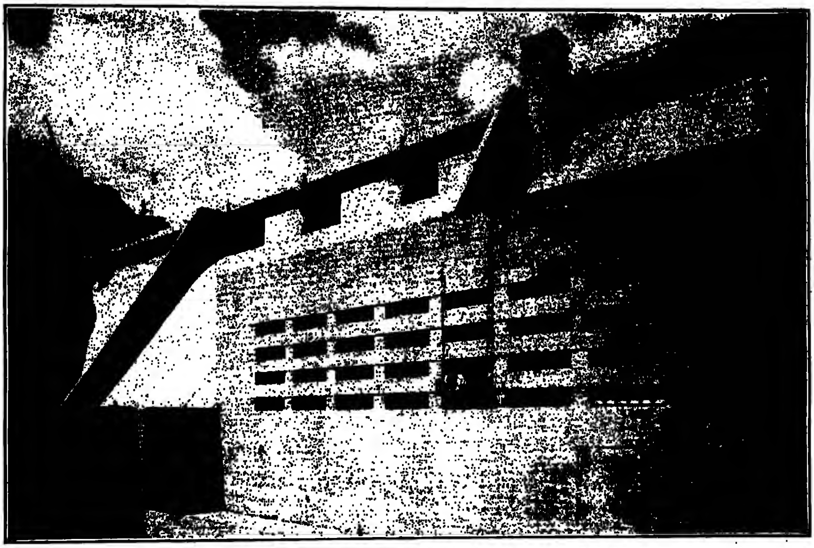


FIG. 9. Paneling on concrete surface at Norris powerhouse, Clinch River, Tennessee. Tennessee Valley Authority. Two units at 66,000 hp, total capacity 132,000 hp, 165-ft head, 112.5 rpm.

If the sash is metal, the maximum number of ventilating sections possible should be supplied and ventilating sections should be placed as near the tops of the windows as possible.

To secure good ventilation at the top of the building, powerhouses are often provided with ventilators in the roof. There is always the danger, however, that leaks may occur in roof ventilators and water thus reach some of the electrical equipment. For this reason ventilators should never be placed directly over electrical machinery.

A better plan for ventilation close under the roof is to place sets of louvers on opposite sides of the building. These can be designed to come directly over windows and thus not affect the general appearance of the building. In fact, they can be treated with a panel effect so that they are hardly noticeable. Either fixed or movable louvers can be used, depending on climatic conditions at the site. (See Figs. 4, 5, and 6.)

A main entrance door big enough to admit the largest piece of machinery must be provided for every powerhouse. This doorway should be at the shore end of the powerhouse or in the side wall next to the shore end of the building (Fig. 3). The working bay should also be at this end of the powerhouse, immediately in front of the doorway.

Either a sliding, folding, or rolling door can be used, each type having its advantages. The sliding door and rolling door are easier to handle but hard to make tight. This difficulty is a disadvantage in northern climates.

The large entrance door should be provided with a wicket door unless it is advisable to have some smaller entrances in addition to the large one. Such smaller entrances are often a convenience in giving direct access to the top of the head wall or the raking platform.

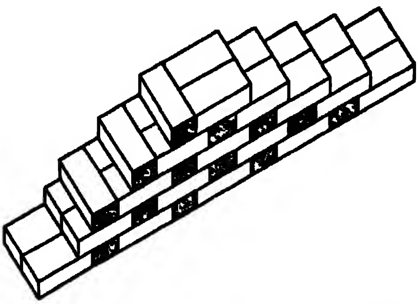


FIG. 10. Brickwork laid up in Flemish bond.

8. Floors. Floors for the substructure are discussed in Section 17 of Chapter 36, and this section applies equally to the floors in the superstructure.

The floors must be designed to support any machinery and materials that may be placed upon them during construction. As in most buildings, the floor is likely to be more heavily loaded during construction than subsequently.

It is rarely safe to design powerhouse floors for less than a 300-lb-per-sq-ft live load, and it is not often necessary to design them for more than a 600-lb-per-sq-ft live load. By careful planning it is sometimes possible, on account of the large size of machinery, to design the girders for the full live load determined upon, reduce this somewhat in the design of the beams, and reduce it still further in the design of the concrete slabs. Caution must be observed in using any such reductions, however.

If there is an outdoor transformer section in connection with the powerhouse, it is usual to have facilities inside the powerhouse for handling and repairing transformers. As a basement is frequently built under the landing bay or part of it, a large hatchway opening into the basement can be made through the main floor. A transformer can then be brought into the powerhouse, lifted by the crane, and lowered through the hatchway until it rests on the basement floor. It then can be dismantled and the core removed by the crane with very little difficulty.

The hatch cover should be framed with structural steel, filled in with concrete, and provided with countersunk rings or holes so that it can be lifted by the crane. To facilitate handling, it can be built in several sections. The hatchway in the floor should be properly framed with angle

curbs, so that the hatch will fit neatly into it and be supported by the necessary beams around the edge. The hatchway must be located so as to be commanded by the crane.

9. Roofs. Pitched roofs are sometimes used with varying degrees of slope, particularly in climates with exceptionally heavy snowfall. However, the flat roof with parapets or with simple overhanging eaves is the type generally adopted. All roofs should be designed for the greatest snow load that may come upon them.

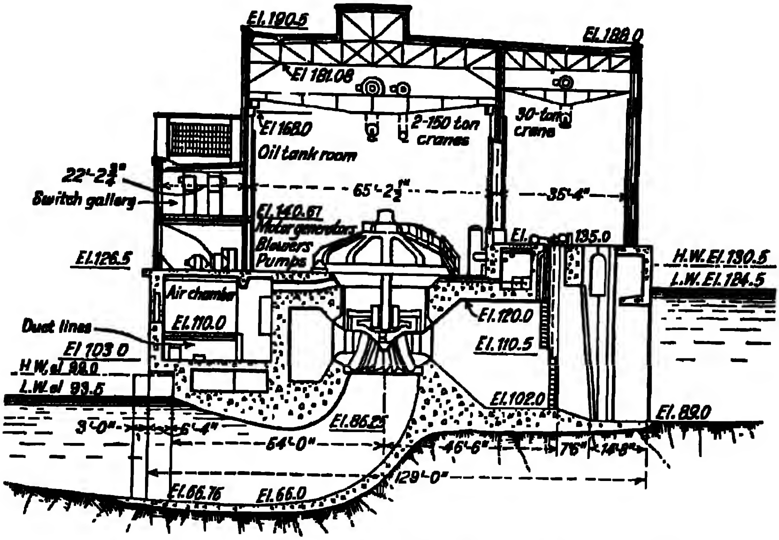


FIG. 11. Cross-section Cedar Rapids powerhouse, Cedar River, Iowa. Electric Light and Power Co. Three units at 1600 hp, total capacity 4800 hp, 10- to 20-ft head.

The cheapest type is the flat roof with overhanging eaves and only enough pitch to give proper drainage. The minimum slope that should be provided is about ¼ in. to the foot. The objection to overhanging eaves in cold climates is that icicles will form along the eaves and may grow to enormous size. On account of the position of the windows in powerhouses, such ice formations are difficult to dislodge, and this is the main reason why parapets should be used in conjunction with flat roofs in such latitudes.

Formerly wood was used to a considerable extent for the roofs of powerhouses. Its advantages are that it is light in weight, cheap, and easily applied. Its disadvantages are that it is not fireproof and is short-lived in comparison with other types of roofs. It has no place in a modern powerhouse.

The roof may be built of simple reinforced-concrete slabs. These should never be less than 3 in. thick, and preferably 4 in. should be the minimum.

There are many other types of fireproof roofs, some of which are patented. These include Hy-rib and similar varieties of reinforced-concrete slabs, concrete plank, hollow clay tile, gypsum slabs, etc. Precast roof members can be threaded to the T-beam purlins, or other type of purlin, thus accelerating the construction of the roof.

Whatever type of roof slab is used, it must be protected by a waterproof covering. The most common and probably the most satisfactory one is the 3-, 4-, or 5-ply built-up roofing finished with a mopping of tar or asphalt protected with slag or gravel as a finish coat.

FIG. 12. Abbreviated type of construction at Norwood powerhouse. Pee Dee River, South Carolina. Two units at 31,100 hp, 90 rpm. and one unit at 25,600 hp, 75 rpm, giving a total capacity of 87,800 hp.

It is also most important to secure heat insulation in the roof, as otherwise moisture will condense on the underside of the roof and water may drop on the electrical equipment. This insulation can be obtained by a cinder concrete fill on top of the roofing slab, or by hollow clay roofing tile, hollow precast concrete plank, hollow gypsum roofing tile, etc.

If the powerhouse has a parapet, copper or lead counter flashing should always be used. The roofing material should be carried up on the walls under the counter flashing. This should be built into a joint in the brickwork, or into a reglet provided for it in the parapet wall, and carefully turned down over the roofing material so as to make a tight and weatherproof joint. Figure 17 shows this construction.

Considerable trouble has resulted from leakage of rainwater into the walls, which causes unsightly efflorescence and, where freezing occurs, rapid disintegration. Usually this is due to poor workmanship in flashing, pointing, and waterproofing, particularly in the joints of coping stones, or to the

omission of effective drips on copings, which permit the water to follow the surface of the stone back into the joint. Careful workmanship and the use of waterproof mortar help to correct this condition.

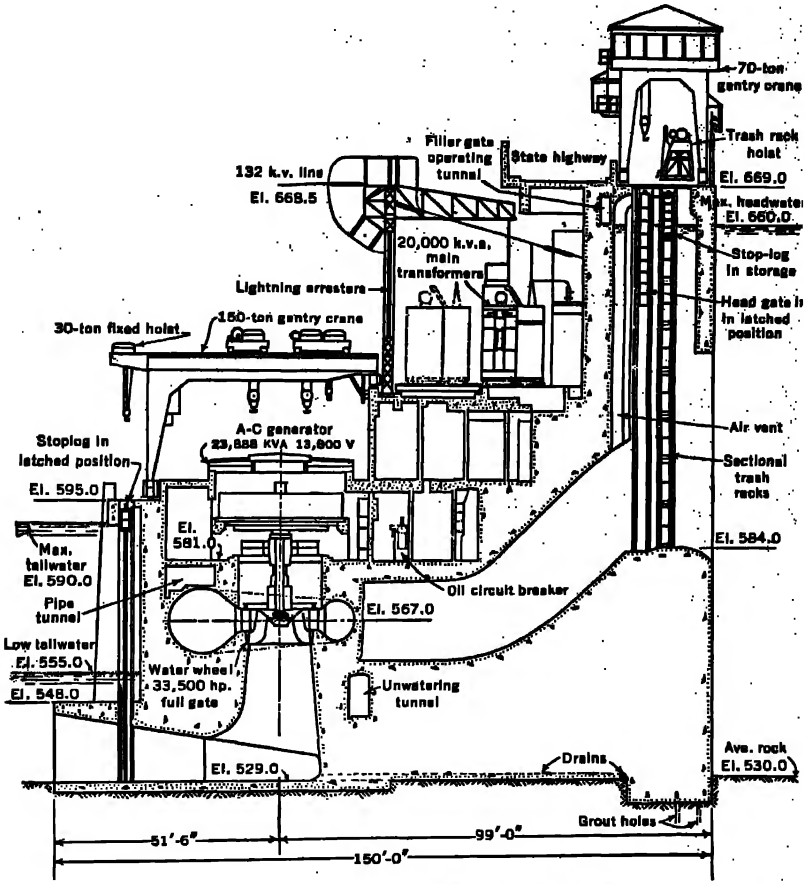


FIG. 13. Cross-section, Bagnell powerhouse, Osage River, Missouri. Union Electric Light and Power Co. of Missouri. Six units at 33,500 hp, total capacity 201,000 hp, 90-ft head, 112.5 rpm, 13,800 volts. (See also Fig. 11, Chapter 11.)

On flat-roof plants where the substation is located on the roof, a slag roof is unwise, but a square-floor tile roof laid on a felt tar membrane wears very well.

The Norfolk plant (Fig. 6) is an example of the use of Spanish tile on a sharply pitched roof.

10. Stairs and Railings. Stairs should always be of steel or concrete. They can be built with channel-iron strings, open risers, and checkered steel-

plate, cast-iron, or bar treads. Sheet-steel treads filled with concrete also make an excellent stair.

Stairways for powerhouses are seldom built on a flatter slope than 45 degrees, although steeper ones are permissible if room is limited, and particularly if they are seldom used. They should always be provided with handrails. If the stairway has a flatter slope than about 45 degrees and if one side is against the wall, a handrail is required on the outside only. If, however, it is steeper than about 45 degrees, or if it is not built against a wall, a handrail on each side is required.

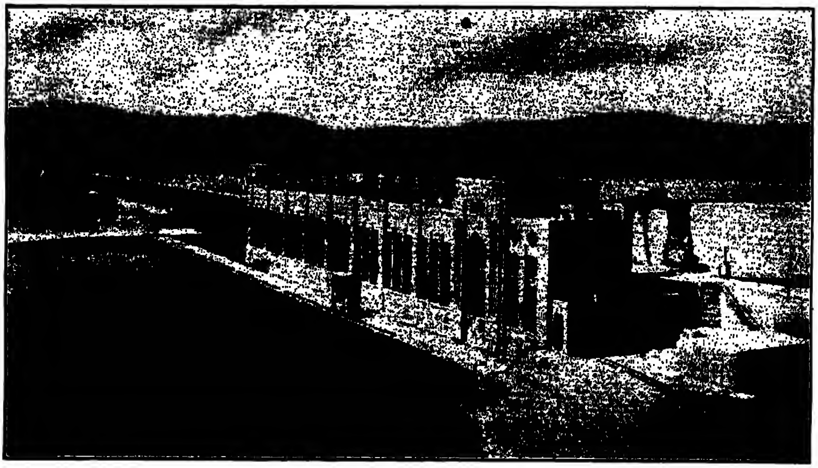


FIG. 14. Brick exterior design, powerhouse, Safe Harbor hydroelectric development, Susquehanna River, Pennsylvania. Safe Harbor Water Power Co. Seven units at 42,500 hp, total capacity 297,500 hp, 55-ft head, 109.1 rpm.

For 45-degree slopes, the treads and risers are usually 8½ to 9 in. For flatter slopes, the treads remain the same and the risers are reduced in height to correspond to the slope. For steeper slopes, the risers may be increased to 9½ or 10 in. and the treads reduced accordingly.

The surfaces of concrete stairs should always be finished rough with a wood float, or safety treads should be used. There are several excellent types of all-pressed-steel stairs on the market, and these can often be used economically. One type of stairway that is generally free from hazard is the pressed-steel riser and framing with a sandy slate tread.

Railings should always be used to protect all openings, stair wells, platforms, etc., both inside and outside the powerhouse. A double rail should be used, the first 22 in. above the floor, and the second 20 in. above the first, making a railing 3 ft 6 in high. One-and-one-quarter-inch black iron pipe is ample for all railing. Standards should be well anchored to concrete by means of flanges with not less than 4 bolts ½ in. by 6 in. long. Standards should be not more than 8 ft 0 in. apart.

All stair wells and edges of galleries should be provided with a concrete o steel toe board 6 in. high above the finished floor level.

11. Water Supply. The water supply for the bearing and transforme cooling systems and for general use is obtained from the nearest continuou source. For high-head developments the supply is from the penstocks; fo low-head plants it is taken directly from the intake

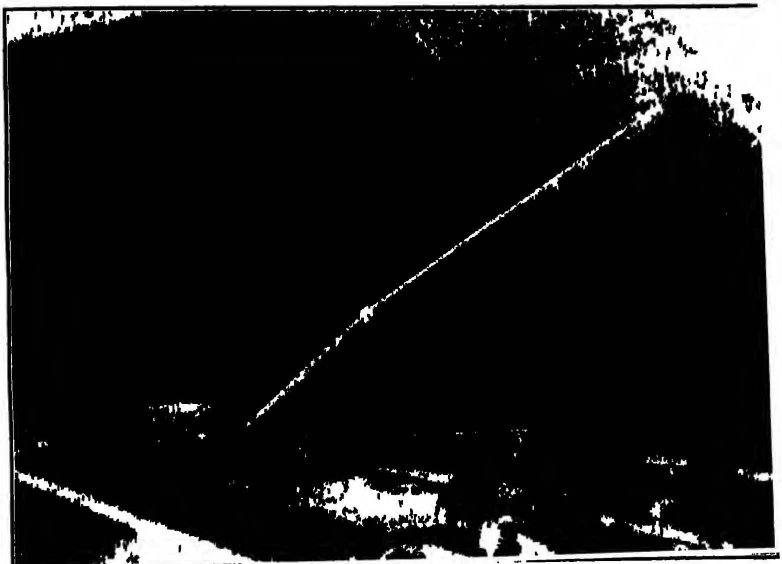


FIG. 15 General view of the Nantahala development and powerhouse, Franklin, North Carolina Nantahala Power and Light Co One unit at 60,000 hp, 925-ft head, 450 rpm

In high-head developments, a header of ample size should be connected to each penstock and provided with the necessary valves so that water will be available even if all the penstocks but one are unwatered. For the same reason, the inlet of the water supply at the intakes of low-head developments should be in an independent bay, or else there should be one in each main bay with a header interconnecting them. A connection to a city water supply, if available, is desirable as an added safeguard; if the river water is extremely silty at times or otherwise unsuitable, the use of the city supply may obviate the necessity for an expensive filter plant, that has had to be installed in some instances

Usually the turbine manufacturer provides an independent source of supply for the bearing cooling system of each turbine, direct from the wheel casing of the respective unit

In a low-head plant, pumps are often needed for the water supply if the head is not sufficient for all purposes. If no alternative water supply is

available, such pumps must be installed in duplicate to insure an uninterrupted flow of water if one pump is out for repairs. The suction line to these pumps should have a foot valve and strainer on the intake end.

Water connections should be provided at convenient points around the powerhouse. The supply pipes should not be less than 2 in. If desired, they can be bushed down to ¾ in. and equipped with ordinary angle valves from

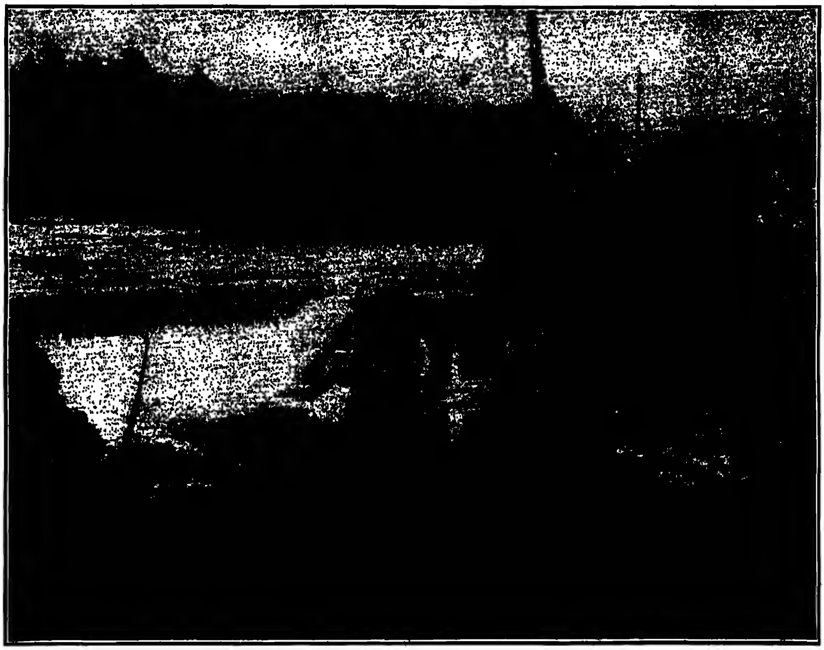


FIG. 16. Powerhouse with stiffleg derrick instead of crane, Winooski Gorge Plant No. 18, Vermont. Green Mountain Power Corp. One unit, 4325 hp, 35-ft head, 180 rpm, 2300 volts.

which to fill pails. Such connections should not be placed near electrical equipment; and it is convenient to have floor drains close by, to take care of any drip or splash from the valves. All such outlets should have threaded ends for the attachment of hose connections and reducers.

All piping installed for water supply should be galvanized genuine wrought-iron pipe, with galvanized malleable- or cast-iron fittings. The first cost of wrought-iron pipe is higher than that of steel pipe, but it will outlast steel pipe and in the end is more economical.

Provision should be made for straining the water supply either by screens at the intake or strainers in the supply line from the penstock connections. It must be possible to clean the screens or strainers without shutting off the supply of water. A duplex strainer will permit continuous water supply even

when one half is being cleaned Under conditions of high river flow, continuous strainer cleaning is mandatory. The newer rotary screens will provide cleaner water than the stationary types.

River-water systems should be designed as emergency or back-up supplies for fire protection Under no circumstance should any connection be made between the river-water system and the drinking-water system Current thought regards the Mulsifyre System or Fog Nozzles as the best approach to fire protection even though the electrical equipment is energized.

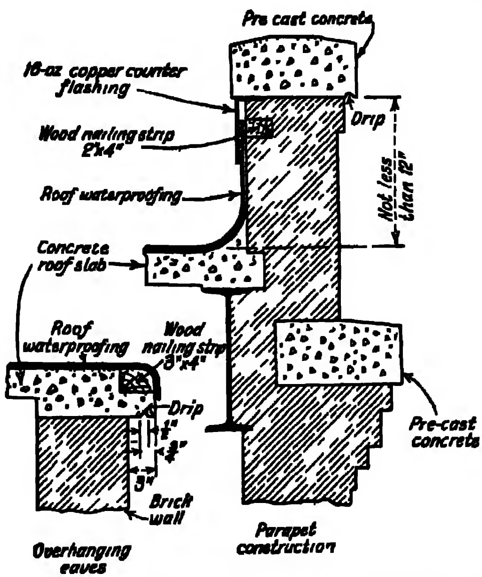


Fig 17 Roof details Overhanging eaves and parapet construction

12. Drainage. If a parapet type of roof is selected, the roof drains should be of the most approved make, as leaks are a great menace to electrical equipment All roof drains must be provided with strainers, preferably of the basket type Leaders should be galvanized-steel or wrought-iron pipe They should drain into the draft tube or tailrace.

To prevent freezing, all piping in cold climates must be kept inside the building and as far from the outside surfaces of walls as possible Piping should not be buried in concrete but should be insulated for the temperature conditions to which it will be subjected All pipes running into the tailrace should preferably extend well under low tailwater, as otherwise the splash of water may plug the outlet of the pipe with ice.

All drains should be run in as straight lines as possible to avoid plugging and cleanouts should be provided wherever necessary to facilitate clearing any drainpipe that becomes plugged.

All floors should have drains to carry off water when the floors are washed. Such drains should be placed away from electrical equipment, and floors should be slightly sloped so that water will drain away from all apparatus.

13. Powerhouse Crane. The traveling crane is generally the largest and heaviest piece of auxiliary machinery in the powerhouse. Its capacity

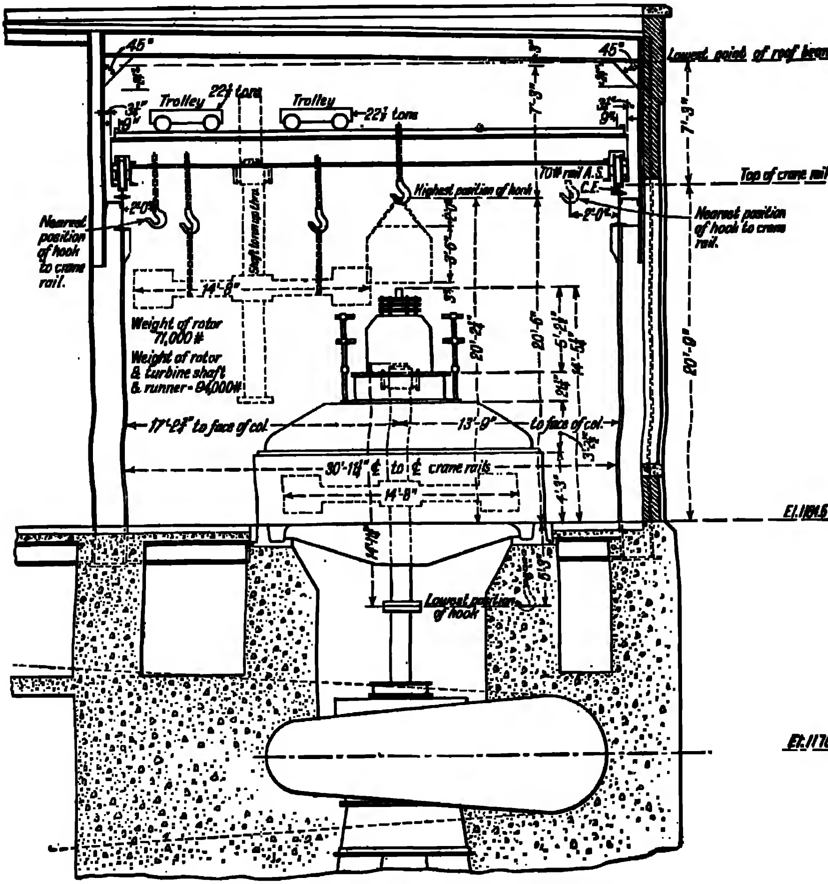


FIG. 18. Typical crane clearance diagram.

and therefore its weight are determined by the heaviest load that it must handle. However, as it handles such a load at only rare intervals, when the runner and its attached rotor are being set, and as this is done with extreme care, the crane in a hydroelectric powerhouse is never subjected to as severe service as a foundry or machine shop crane. Therefore, a lighter type of crane can be installed than would be advisable for the heavier service.

Either a one-trolley (Fig. 2) or a two-trolley crane (Figs. 13 and 18) can be used. The one-trolley crane is by far the cheaper, but, on the other hand, it requires a higher powerhouse for sufficient clearance. With two trolleys, the shaft of either the rotor or the runner can be hoisted up so as to pass up between the trolleys and the crane bridge girders, thus materially reducing the height of the powerhouse necessary to obtain clearance (see Fig. 18). This cannot be done with a single trolley. On the other hand, it is much easier to adjust the rotor and runner with a single trolley, as there is only the one hoist to control. If the height of the powerhouse is to be determined by the clearance necessary to carry the rotor or the runner, with its shaft, over the other units, then the two-trolley crane will probably be the more economical. If, however, for some other reason, the powerhouse has to be built higher than otherwise would be necessary, and there is clearance for a one-trolley crane, then this installation is usually the cheaper.

A high-speed, low-capacity auxiliary hook on the main crane will expedite the jobs that the crane must do from time to time. It is most important to design the crane so as to avoid any overloading. In this connection, consult the various codes concerning crane design.

In the design of the steel in the superstructure, at least two roof trusses and their supporting columns must be adequate to safely care for the assembly of the crane. Where heavy cranes are to be installed, this is most important.

In setting the runner and rotor of a vertical unit with the crane, the procedure is as follows:

The runner is lowered into place. In the Francis type of runners, the turbine casing has a ring on which the runner can rest. In the propeller type of runner, it is generally necessary to use blocking on which to rest the runner, although the turbine manufacturers sometimes provide special lugs which are bolted to the casing and on which the runner can rest.

After the generator stator is erected, the rotor is lowered into position and leveled up temporarily on blocking or on the generator brakes. This generally leaves from 1- to 1½-in. clearance between the generator-shaft coupling and the turbine-shaft coupling. Next the coupling bolts are drawn up and the runner raised to its final position. The thrust bearing is then installed and adjusted, and the temporary support of the rotor is removed.

A clearance diagram similar to that shown in Fig. 18 should always be made and submitted for approval to the crane, turbine, and generator manufacturers.

14. Types of Gantry Cranes. The usual gantry crane has legs of equal length and a hoisting trolley that travels on the bridge between the legs. In gantries common in the abbreviated types of powerhouses, there have been various modifications of this type. Where the generators are placed close under the downstream face of the dam, one leg of the gantry may be a short stub leg traveling on a rail set on top of the dam (Fig. 12) or on a

bench provided for it on the downstream face. In other installations the gantry legs have been supported by the low walls of the abbreviated powerhouse superstructure (Fig. 13).

In some developments, head gates are controlled by a cantilever arm on the upstream side, and stoplogs or tail gates by one on the downstream side (see Fig. 7). In one or two cases, the cantilever arm has been used to hoist materials from barges floated to the powerhouse site, both during construction and for emergency purposes afterwards. This method is desirable where it might be difficult or impossible to gain direct access to the site by rail. The drawbacks are that it necessitates a dock and loading facilities near by and the maintenance of a barge and towing equipment available on short notice. The barge must be large enough to transport the largest piece of equipment in the power plant.

Gantry cranes are more expensive than ordinary cranes of equal capacity. On account of the large units now being installed, gantries of very great capacity are required. The gantry of the powerhouse at Mitchell Dam on the Coosa River has a capacity of 125 tons on the main bridge with 70 tons capacity on both cantilever arms. At Wheeler Dam powerhouse of the Tennessee Valley Authority, where there are several gantry cranes, the largest one having a capacity of 300 tons commands the units. This is a simple gantry without cantilever arms.

15. Telephone and Communication. The telephone booth should be enclosed with terra-cotta, brick, or concrete walls and ceiling, and provided with a good single door, or preferably a double door, to keep out as much noise as possible. It should also have a false wooden floor supported on glass insulators, to reduce as much as possible the chance of a heavy shock to anyone using the telephone.

In a large plant, where a number of telephones are necessary, a decision must be made between operating its own telephone system and leasing from the Bell System. Leasing is usually preferable. In a large plant, a public-address system or a call system expedites operating routine. In remote locations, carrier-current or short-wave radio may prove the least expensive.

16. Heating. In cold climates, the question of heating a powerhouse is one that must be considered. At ordinary low temperatures down to freezing, enough heat is given off by the generators to keep the body of the building comfortably warm, but at lower temperatures, or in a large powerhouse where a number of the units may be shut down at the same time other means must be provided for supplying enough heat to keep at least a part of the powerhouse, where the operator is stationed, reasonably warm (68 degrees Fahrenheit). The value of heating to operating-personnel morale should never be underrated. For this reason, in all powerhouses in cold latitudes, one or more chimneys with proper flues (not less than 8 in. by 8 in.) should be provided. These chimneys should be located at convenient points so that stovepipes can be easily connected to them. All chimneys should have terra-

cotta flue lining surrounded with at least 8 in of brick. They need not be much higher than the roof line because of the chimney effect at the dam site.

In some cases, the switchboard is located in a separate room or enclosed in a glass partition, and this enclosure is heated with electric heaters, small stoves, or wall-mounted unit blower heaters. Another method is to keep the operator's desk and chair in a small, heated, portable enclosure in front of the switchboard.

EQUIPMENT AND OPERATION

CHAPTER 38

HYDRAULIC TURBINES

By Arnold Pfau and the late Dr. William Monroe White

1. Introduction. The computation of power and of the head on the turbine has been considered in Chapter 9, "Head, Power, and Efficiency." On this subject the reader is also referred to Section 22 of this chapter. Power may be developed from water by three fundamental processes: by action of its weight, of its pressure, or of its velocity; or by a combination of any or all three. In modern practice the Pelton or impulse wheel is the only type which obtains power by a single process, the action of one or more high-velocity jets. This type of wheel is usually found in high-head developments.

2. Progress in Design. As shown by Fig. 1 there has been practically no increase in the efficiency of hydraulic turbines since about 1925, when maximum efficiencies reached 93% or more [1]. As far as maximum effi-

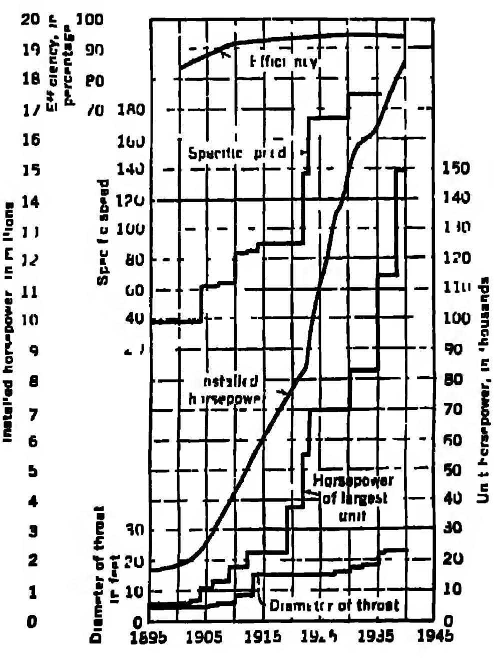


FIG. 1. Progress chart of hydraulic turbine development. (After Fig 1, "Hydro Generation of Energy," by Frank H. Rogers, *Trans. A.S.C.E.*, Vol. 104, p. 952, 1939.)

ciency is concerned, the hydraulic turbine has about reached the practicable limit of development. Nevertheless, in recent years, there has been a rapid and marked increase in the physical size and horsepower capacity of individual units, as may also be seen from Fig. 1 and Table 1.

TABLE 1

Representative Hydraulic Turbines

Station	Location	Type	Manufacturer	Rated Hp One Unit	Rated Head	Discharge Diameter, of Runner, inches	Speed, rpm	Specific Speed (One Jet for Impulse Wheel)
Apalachia	Tenn.	Francis	I	53,000	360	108	225	33.0
Boulder	Nev.	Francis	A	115,000	510	130	180	25.2
			I	115,000	510	130	180	25.2
Claytor	Va.	Francis	A	26,000	110	129	138.5	62.7
			N	26,000	110	128.25	138.5	62.7
Conowingo	Md.	Francis	A, I	54,000	89	194	81.8	69.5
Dnieprostroi	Russia	Francis	N	84,000	116.5	226.50	88.2	67.3
Grand Coulee	Wash.	Francis	N	150,000	330	172.75	120	33.3
			A	103,000	330	171	138.5	31.75
Hiwassee	N. C.	Francis	N	80,000	190	165.19	120	48.1
Ixtapantongo	Mex.	Francis	I	39,000	1,028	48.62	600	20.3
Martin	Ala.	Francis	A	45,000	145	136	120	50.6
Marshall Ford	Tex.	Francis	I	27,000	120	128.5	144	59.5
Nantahala	N. C.	Francis	N	60,000	925	59.94	450	21.6
Niagara Falls No. 21	N. Y.	Francis	A, I	70,000	214	176	107	34.5
Norris	Tenn.	Francis	N	66,000	165	165.19	112.5	48.9
Osage	Mo.	Francis	A	33,500	90	133	112.5	74.5
Pit River No. 5	Calif.	Francis	P, A	50,000	587	77.5	300	23.2
San Francisquito No. 2	Calif.	Francis	N	21,000	515	50.31	428	25.3
Santeetlah	Tenn.	Francis	S	33,000	660	53.56	450	24.5
Wilson	Ala.	Francis	I	30,000	95	150.5	100	58.4
			N	30,000	95	179	100	58.4
			A	35,000	95	133	100	65.8
Bonneville	Ore.	Kaplan	S	74,000	60	280	75	122.3
Buzzards Roost	S. C.	Kaplan	I	7,400	60	92	240	124
Chickamauga	Tenn.	Kaplan	I	36,000	36	264	75	161
Great Northern Paper Co.	Me.	Propeller	I	3,050	28	100	171.5	147.3
Green Island	N. Y.	Propeller	A	2,200	13	156	80	152
Fort Loudoun	Tenn.	Kaplan	I	50,000	65	222	105.8	128.2
Guntersville	Ala.	Kaplan	S	34,000	36	265	69.2	145
Marmet	W. Va.	Propeller	N	7,600	23	177	90	155.8
Ohio Falls	Ky.	Propeller	A	13,500	37	180	106	135
Pickwick Landing	Tenn.	Kaplan	A	48,000	43	292	81.8	163
Rock Island	Wash.	Propeller	A	21,000	32	212	100	190
Safe Harbor	Pa.	Kaplan	I, S	42,500	55	220	109	149.3
Shannon	Ireland	Kaplan	Foreign	38,000	106.5		167	89
Watts Bar	Tenn.	Kaplan	I	42,500	52	234	94.7	139
Wheeler Units Nos. 3 and 4	Ala.	Propeller	I	45,000	48	264	85.7	144.1
Balch	Calif.	Impulse (1 jet)	A	2 × 20,000	2,243	115 pitch dia.	360	3.3
Big Creek No. 1	Calif.	Impulse (1 jet)	P	2 × 17,500	1,900	121 pitch dia.	300	3.16
Big Creek No. 2A	Calif.	Impulse (1 jet)	A	2 × 28,000	2,200	162 pitch dia.	250	2.79
Bucks Creek	Calif.	Impulse (1 jet)	P	2 × 15,000	2,400	113 pitch dia.	450	2.75
Caribou	Calif.	Impulse (1 jet)	A	2 × 15,000	1,008	155 pitch dia.	171	3.7
Feather River	Calif.	Impulse (1 jet)	P	2 × 15,000	2,350	89 pitch dia.	450	3.37
Glenville	N.C.	Impulse (1 jet)	A	2 × 15,000	1,150	117 pitch dia.	257	4.7
San Francisquito No. 1	Calif.	Impulse (1 jet)	A	2 × 16,100	800	176 pitch dia.	143	4.25
Tiger Creek	Calif.	Impulse (1 jet)	P	2 × 18,000	1,190	129.5 pitch dia.	225	4.33
Toro Negre No. 2	Puerto Rico	Impulse (1 jet)	S	1 × 2,700	630	50 pitch dia.	300	5.0
Danville	Va.	(Vertical) impulse (2 jets)	S	1 × 4,550	690	72.1 pitch dia.	450	6.07
Waiau	Hawaii	Impulse (2 jets)	P	2 × 1,125	400	41.5 pitch dia.	400	5.3

Manufacturer:
A = Allis Chalmers Manufacturing Company.
I = Baldwin Southwark—I.P. Morris.
N = Newport News Shipbuilding and Drydock Company.
P = Pelton Waterwheel Company.
S = S. Morgan Smith Company.

Most noteworthy, however, has been the development of the propeller- and Kaplan-type turbines, which largely accounts for the increase in specific speed shown in Fig. 1 [4, 6, 7, 9, and 10]. This development is of major economic importance, having materially decreased the total cost of low-head installations and made economically feasible many low-head projects which were previously impracticable. This is because the higher speeds practicable with the Kaplan- and propeller-type turbines mean higher-speed, lower-cost generators, and somewhat smaller space requirements in the power plant for a given capacity.

In addition there has been improvement in the details of design of the various elements of the power plant in the direction of simplification and the reduction of operating labor. There has also been considerable research into the cause and prevention of cavitation, which allows the advantages of higher specific speeds to be obtained at higher heads than formerly were considered advisable. The net effect of this progress with larger units, higher specific speed, and simplification and improvements in design has been to retain for the hydraulic turbine the important place which it has long held at one of the most important prime movers.

3. Types of Hydraulic Turbines. Hydraulic turbines may be grouped in two general classes: the impulse type which utilizes the kinetic energy of a high-velocity jet which acts upon only a small part of the circumference at any instant, and the reaction type which develops power from the combined action of pressure and velocity of the water that completely fills the runner and water passages. The reaction group is divided into two general types: the Francis, sometimes called the reaction type, and the propeller type. The propeller class is also further subdivided into the fixed-blade or propeller type, and the adjustable-blade type of which the Kaplan is representative.

In Table 1 is given a representative list of hydraulic turbines many of which are shown in the figures. The type, head, discharge diameter, revolutions per minute, and specific speed appear in the table.

Impulse Wheels. With the impulse wheel the potential energy of the water in the penstock is transformed into kinetic energy in a jet issuing from the orifice of a nozzle. This jet discharges freely into the atmosphere inside the wheel housing and strikes against the bowl-shaped buckets of the runner illustrated in Fig. 2.

Impulse wheels (Fig. 2) are used at heads of 800 ft or more, although they may be used at lower heads, depending on the horsepower capacity involved. Usually not more than one or two jets are applied to the circumference of the runner or bucket wheel. The specific speed suitable for a given head and capacity of unit is much lower than that for the Francis or for the propeller type.

One of the largest impulse wheels, from the standpoint of physical size, operates at the San Francisquito No. 1 Plant in California. It is of the double-runner overhung single-jet type developing 32,200 hp under 800-ft head at 143 rpm (50 cycles) and is now operated without change alternately

at that speed and at 60 cycles (171 rpm), as demand may require. There is practically no loss in output or efficiency at the higher cycles. A high-head plant, the Bucks Creek Plant on the Feather River, California, with a maximum head of 2575 ft, uses impulse wheels. In Switzerland a few units operate under 5000 ft or more.

At each revolution the bucket enters, passes through, and passes out of the jet, during which time it receives the full impact force of the jet. This produces a rapid hammer blow upon the bucket. At the same time the

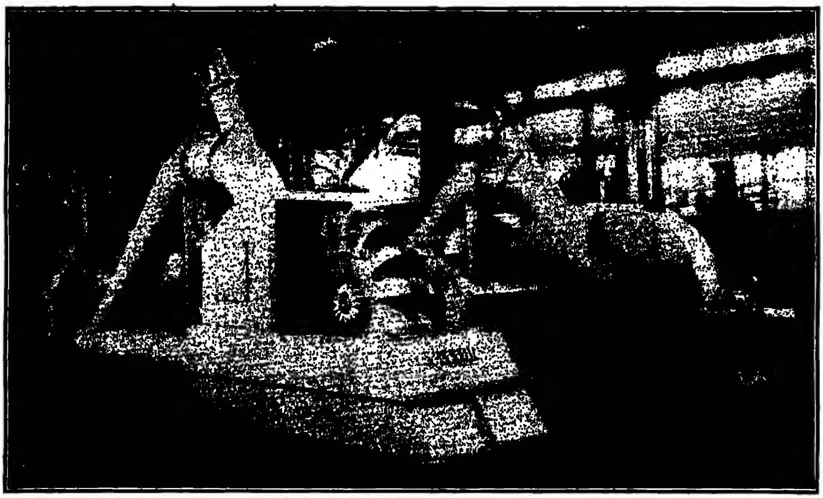


FIG. 2. Waiau power plant, Hawaii; 2250 total hp, Pelton double-overhung, double-nozzle impulse wheels, 400-ft head, 400 rpm. (See Table 1.)

bucket is subjected to the centrifugal force tending to separate the bucket from its disk. On account of the stresses so produced and also the scouring effect of the water flowing over the working surface of the bowl, material of high quality of resistance against hydraulic wear and fatigue is required. Only for very low heads can cast iron be employed. Bronze and annealed cast steel are normally used.

To permit ready replacement of damaged buckets, they are usually individually bolted to disks, either singly or in segments of two or more, if the pitch has become so limited that not enough bolts could be provided to fasten a single bucket safely. Single buckets permit ready and high finishing of the working surface of the bowls, thereby assuring highest efficiency.

Francis Runners. With the Francis type the water enters from a casing or flume with a relatively low velocity, passes through guide vanes or gates located around the circumference, and flows through the runner, from which it discharges into a draft tube sealed below the tailwater level. All the water passages are completely filled with water, which acts upon the whole circumference of the runner. Only a portion of the power is derived from the

dynamic action due to the velocity of the water, a large part of the power being obtained from the difference in pressure acting on the front and back of the runner buckets [2]. The draft tube allows maximum utilization of the available head, both because of the suction created below the runner by the vertical column of water and because the outlet of the draft tube is larger than the throat just below the runner, thus utilizing a part of the kinetic energy of the water leaving the runner blades.

A comparison of various types of reaction runners of the same power, but of different specific speeds, is shown in Fig. 3. The top three sections show

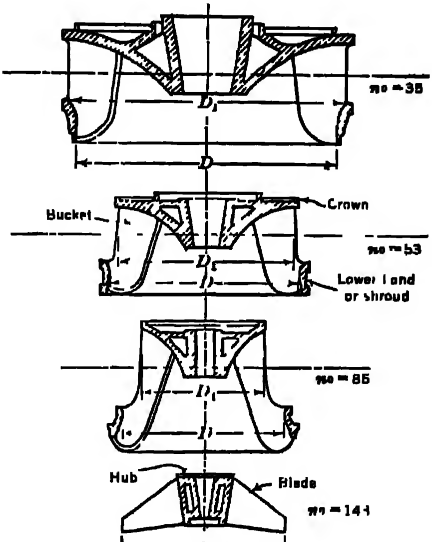


FIG. 3 Comparison of runners of equal power but different specific speeds. D_1 inlet diameter; D, discharge diameter. (Adapted from "Hydro Generation of Power," by Frank H. Rogers, *Trans. A.S.C.E.*, Vol. 104, p. 961, 1939.)

Francis-type runners and the bottom section a propeller-type runner. As the specific speed (see Section 5) is increased, the figure indicates that the flow through the runner changes from radially inward to more nearly axial. Roughly the propeller runner may be considered as a development of the Francis type in which the number of blades is greatly reduced and the lower band omitted. Strictly, however, the process of flow of water from the guide case to and through a propeller is quite different because of the large space, called the whirl chamber, between the stationary guide case and the entrance into the propeller runner.

Figure 4 is a shop view of a partially assembled large Francis runner for the Dnieprostroi plant, Russia, which clearly shows the shape of the buckets, crown, and lower band. Figures 5 and 6 are other examples of Francis runners.

FIG 4 Dnieprostroi plant Russia, 84 000-hp cast-steel Francis runner, 116 5-ft head 88 2 rpm (Newport News) (See Table 1)

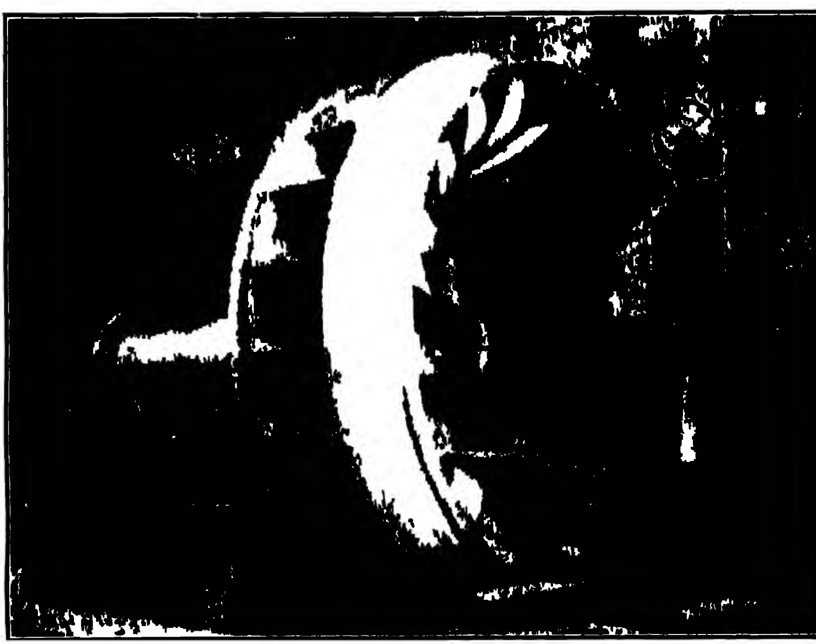


FIG 5 Niagara Falls, New York, 70 000-hp cast-steel Francis runner, 176-in outlet diameter runner bolted to forged flange on main shaft 214-ft head, 107 rpm (See Table 1) (Allis-Chalmers)

Francis runners having a specific speed of 20 to 100 may be constructed of cast iron, cast steel, or bronze, or with cast-iron or cast-steel hubs and rims and plate-steel buckets. For heads above 250 ft, either bronze or cast-steel runners are recommended, as the tensile strength of ordinary cast iron is low and too great a thickness of section would be required. For low-head conditions the plate-steel buckets have been used successfully up to 75 ft. This construction greatly reduces the cost of building the runner, and it is entirely suitable for these conditions as the plate steel is an excellent metal for this purpose and by proper treatment a practically perfect bond can be made between the end of the blades and the cast-iron or cast-steel hub. Plate-steel buckets, being thinner than cast buckets, allow more water to pass through the runner and, consequently, more power to be obtained for the same size of wheel.

FIG. 6. Wilson or Muscle Shoals; 30,000-hp cast-iron Francis runner, 95-ft head, 100 rpm. Shows keyway and taper fit for connection to main shaft. (See Table 1.) (I. P. Morris.)

Propeller Runners. Inherently suitable for low-head developments, the propeller-type unit has effected marked economies within the range of head to which it is adapted. The higher speed of this type of turbine results in a lower-cost generator and somewhat smaller powerhouse substructure and superstructure.

Fixed-blade Type. As shown by Fig. 19 the fixed-blade propeller-type unit has a high efficiency (88% in this case) at a point near full load, but the efficiency drops off rapidly as the load decreases, until, at 40% of full load, the efficiency is only about 50%. The installation of the fixed-blade propeller-type runner should therefore be limited to developments where the units may be operated at a point near maximum efficiency or not at all, and where, under variable head, the load of the unit could be shifted to an output resulting in best efficiency [22].

Figures 7, 8, and 9 show examples of propeller-type runners with fixed blades. Runners of this type have been in commercial use in the United States since 1916.

Materials. Propeller-type runners for low heads and small outputs are sometimes constructed of cast iron. For heads above 20 ft, they are made of cast steel, a much more reliable material. Large-diameter propellers may have individual blades fastened to the hub.

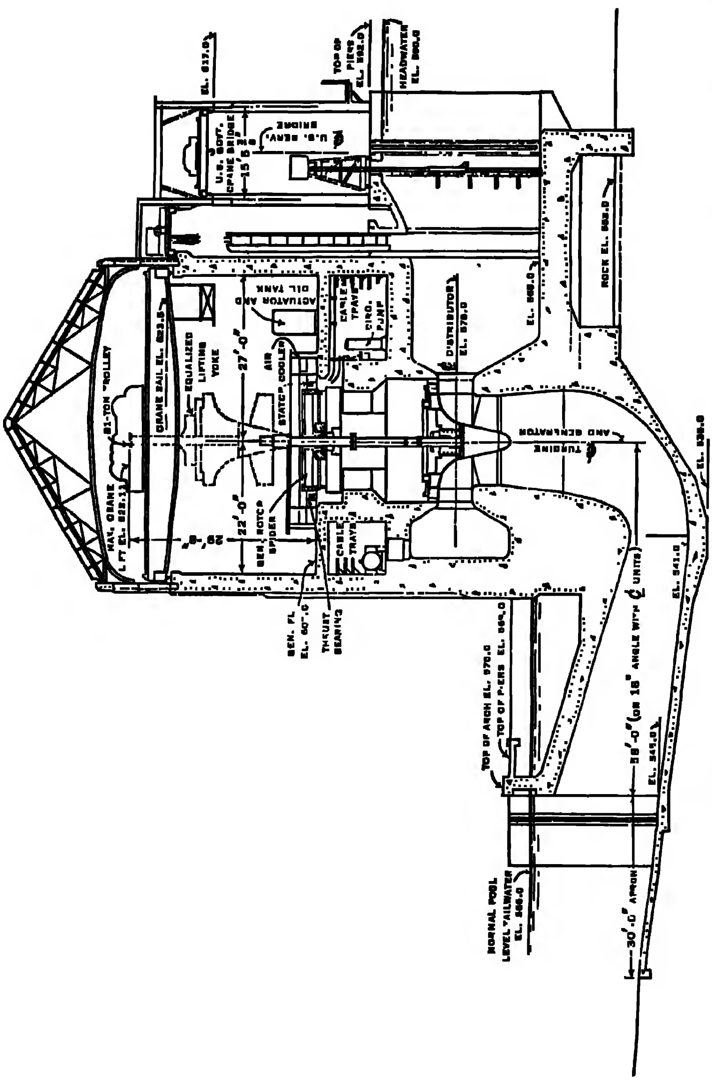


FIG. 7. Marmet plant. West Virginia. Cross-section through 6600-hp fixed-blade unit. (Newport News.) One 7250-hp (I. P. Morris) Kaplan unit, and one 7600-hp (Newport News) adjustable-blade unit have also been installed at this plant (23-ft head, 90 rpm). (See Table 1.)

Adjustable-blade Runners. The adjustable-blade propeller type [4, 6, 9, 10] is a development from the fixed-blade propeller wheel. One of the best-known units of this type is the Kaplan unit, in which the blades may be rotated to the most efficient angle by a hydraulic servomotor. A cam on the governor is used to cause the blade angle to change with the gate position so that high efficiency is always obtained at almost any percentage of full load. Figure 19 shows the characteristically flat efficiency curve of this type of unit. Earlier developments in the United States provided for a maximum adjustment of the blades with turbine at rest to compensate for changes in the head or limitations of discharge (see Fig. 11).

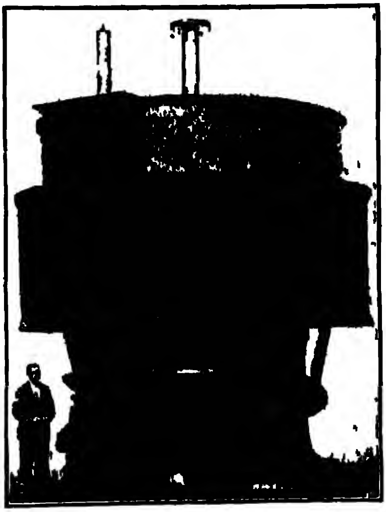


FIG. 8. Norman Plant, Ontario, Canada; 3600-hp cast-iron runner and turbine assembly, 124-in. diameter, 120 rpm, 22.5-ft head and 147 specific speed. Three-blade runner, keyed to main shaft, cast-iron speed ring, plate-steel draft tube and pit liner. (S. Morgan Smith.)

As a step between the two, the so-called motormatic type, which permits a manual or electric adjustment while the unit remains in operation (see Fig. 12), is used in the United States.

Figure 10 is a diagrammatic sketch of a Kaplan unit [7] illustrating in particular how the blade angle is automatically adjusted to obtain high efficiency at any gate opening. Here the hub is cut away showing the spider and linkage which twists the four blades as the servomotor piston within the hollow shaft is raised and lowered by oil under pressure. Oil is admitted below the piston through the hollow piston rod. The sliding cam connected to the gate mechanism which controls the oil to the servomotor piston is also shown in the right-hand portion of the figure.

By reason of its high efficiency at all gate openings, the adjustable-blade propeller-type unit is particularly applicable to low-head developments where conditions are such that the units must be operated at varying load and varying head. Capital cost and maintenance for such units are necessarily higher than for fixed-blade propeller-type units operated at point of maximum efficiency.

Consequently, for a low-head development with fairly constant head and comprising a number of units, consideration should be given to installing fixed-blade propeller-type runners for most of them and adjustable-blade propeller type runners for only one or two units [22]. Then the fixed-blade units could be operated at point of maximum efficiency or not at all

and the adjustable-blade units could take the necessary variation in load. Such an arrangement is particularly applicable to a large power system containing a multiplicity of units.

Figure 11 shows the rotary element of one of the Nagler propeller-type [4] hand-adjustable six-blade units at the Rock Island Plant near Wenatchee,

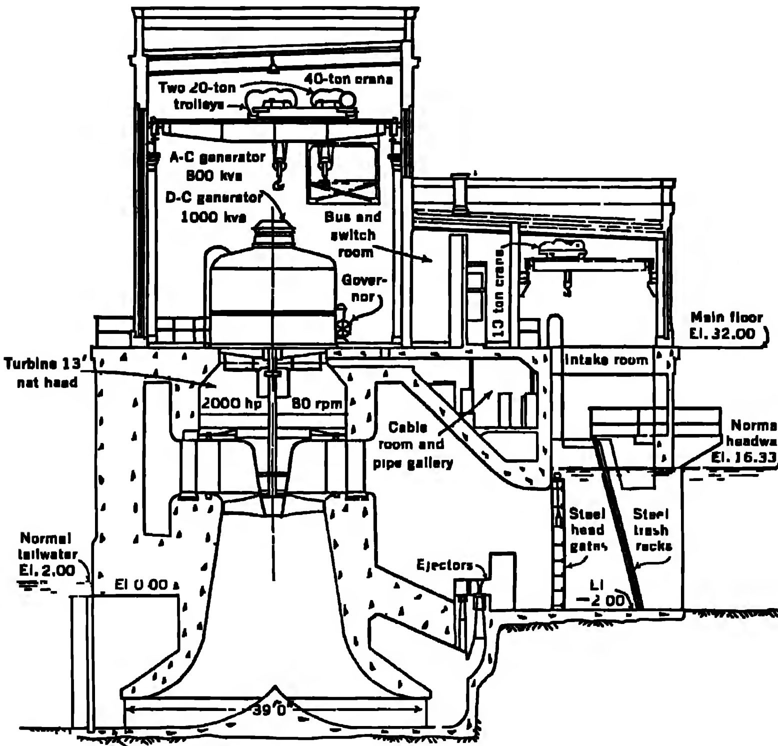


FIG. 9 Green Island plant, Troy, N. Y.; 2200-hp cast-steel propeller turbines; four blades cast separately, bolted together, and to forged flange of main shaft; 13-ft head, 80 rpm. A pioneer propeller plant in the United States. (See Table 1.) (Allis-Chalmers)

Washington, installed in 1930. The six propeller blades are mechanically adjusted by turning a shaft located in the main shaft flange when the turbine is at rest, requiring a total time of about 3 minutes from old load carried to new load carried. The figure shows the heavy journal bearing which must be provided to carry the heavy thrust load of the water and still allow the blades to rotate to change the pitch. Figures 12 and 13 show examples of motormatic and of governor adjustable-blade propeller-type turbines.

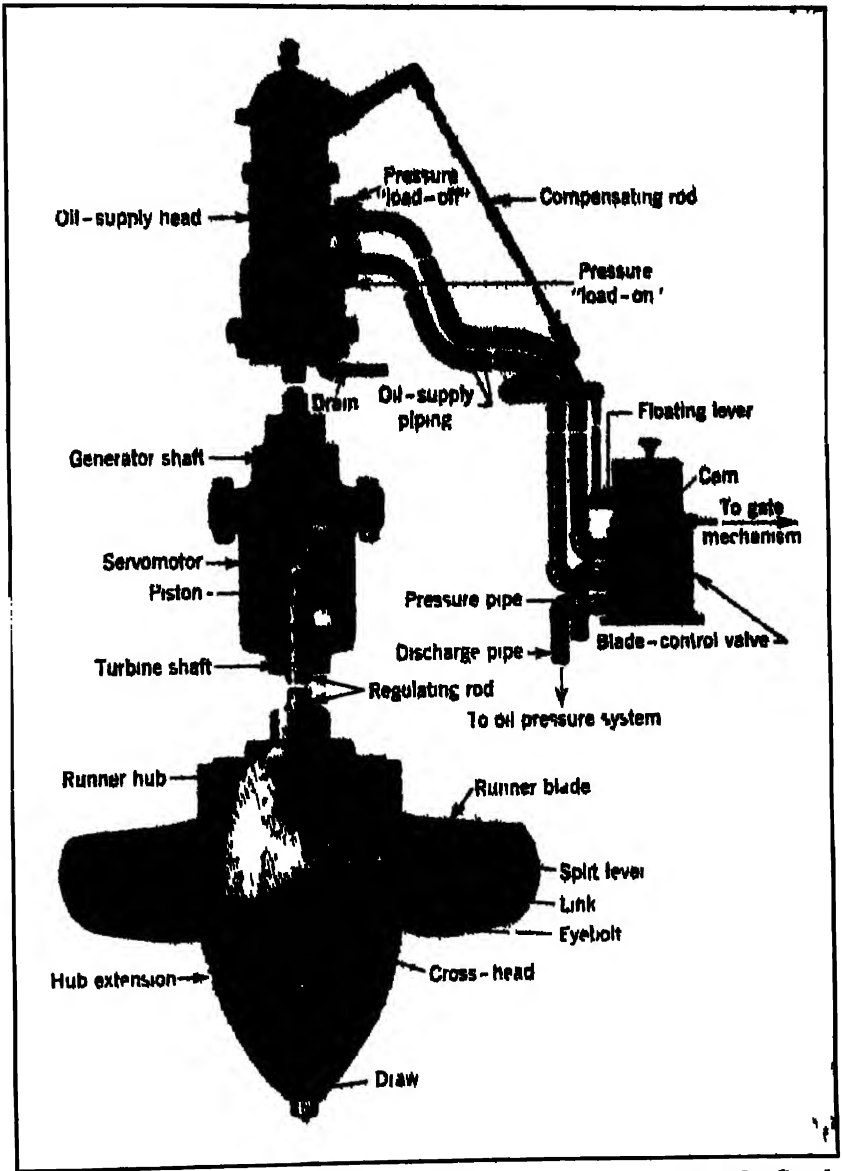


FIG 10 Diagrammatical arrangement of the four major components of a Smith-Kaplan runner and control (S Morgan Smith Company)

The Terry propeller turbine [10] shown in Fig. 14 is another form of the adjustable-blade propeller-type unit. It has movable blades pivoted slightly ahead of the normal center of pressure so that the flow of water past the blades tends to open them against the resistance of a balanced piston within the hub which is acted on by water pressure. This causes the blades to

FIG. 11. Rock Island plant, Washington; 21,000-hp, six-blade, adjustable Nagler-type runner, 32-ft head, 100 rpm. (See Table 1.) (Allis-Chalmers.)

assume automatically the most efficient angle for the flow of water admitted by the gates. The blades are pivoted on roller bearings and are interconnected in a grease-filled hub to the piston rod and balanced piston.

One of the advantages of this type of runner is that the blades open dur-

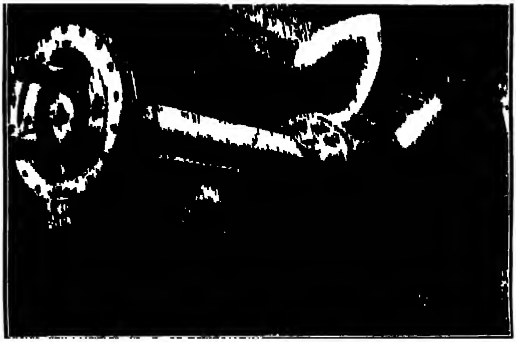


FIG. 12. Montreal Island plant, Canada; 12,000-hp, 196¼-in. adjustable-blade runner, 26-ft head, 85 rpm. (Allis-Chalmers.)

ing starting, thus reducing the starting load on the thrust bearing. They also open in case of overspeed, thus reducing the amount of overspeed.

4. Selection of Type of Turbine. Section 3 has briefly described the various modern types of turbines and the conditions under which they are designed to operate. The problem confronting the engineer is to determine

the type and setting that will result in the most economical development under the given existing conditions. Each type of turbine has its limitations in application. The *impulse wheel* is, in general, used for very high heads, above 800 ft, although, in special cases where it is used to drive plant service equipment requiring small horsepower capacity, this type of wheel is used at heads much lower than 800 ft. By reason of its lower costs of parts exposed to natural wear and tear it may be preferred to other types in cases where there is an excess of water.

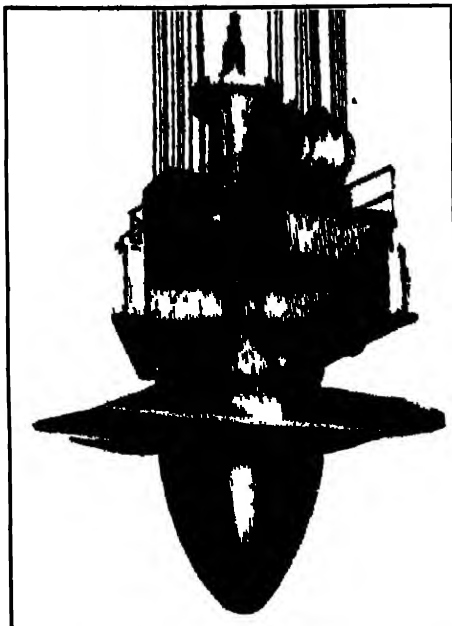


FIG. 13. Bonneville plant, Oregon, 74 000-hp Kaplan turbine, 60-ft head 75 rpm. (See Table 1.) (S. Morgan Smith.)

The Francis type can be efficiently and economically employed at heads ranging from 50 to 800 ft. One installation has been in operation for several years under a head of 960 ft.* Under such high heads the unit should be operated near best point of efficiency (head) so as to avoid excessive hydraulic wear, especially at low part load. Operating water free of sand and chemicals is indispensable. There are still many plants installed long before the development of the propeller type for heads down to 8 ft for direct coupling to generators, or for even lower heads, employing bevel gears or belt, or rope drives, to higher-speed generators or transmission-line shafts.

* At the Nantahala Development in North Carolina, the Aluminum Company of America in 1942 installed a 60 000-hp Francis unit under a rated head of 925 ft.

The propeller type is generally used for low-head developments of heads from 10 to 80 ft, but units of this type have been used under heads somewhat exceeding 100 ft. The maximum head for which the propeller type can be

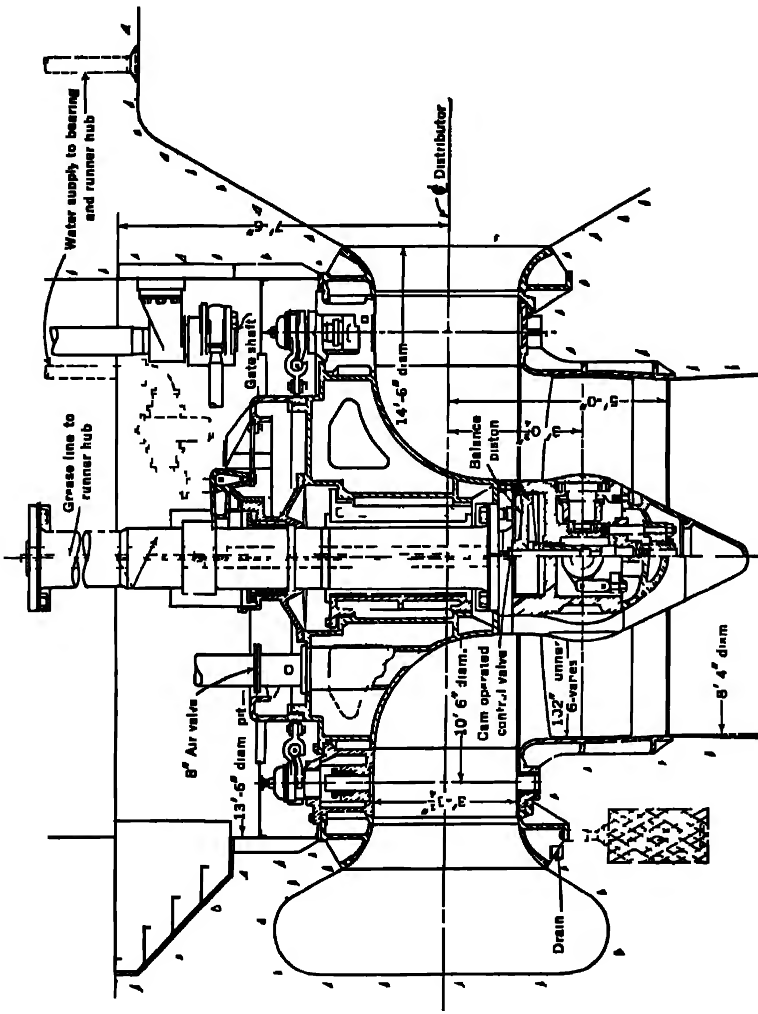


FIG 14 Austin plant Texas; 10 000-hp Terry adjustable-blade propeller runner. 61-ft head. 200 rpm (Newport News)

used is, among other factors, dependent on the economic limit of preventing cavitation [5] by lowering the setting of the wheel as discussed in Section 7 The adjustable-blade propeller units are used in the same range of heads as the fixed-blade propeller type and are subject to about the same limitations.

Other factors affecting selection of a propeller type as against the Francis type are:

(*a*) The propeller has a higher runaway speed, thus involving a correspondingly more expensive design of generator.

(*b*) The hydraulic thrust is higher, requiring a more costly thrust bearing.

(*c*) Figure 15 clearly indicates that with increasing head the suitable specific speed becomes lower. For instance, for heads greater than 100 ft it is below 90 This brings it nearer to a Francis turbine speed so that the over-all cost of a propeller turbine and generator combined as compared with that of a Francis turbine and generator combined may not offset the increased cost of excavation, as outlined in Section 7.

(*d*) On account of the long passageway from the end of guide vanes to runner vanes of a propeller, it cannot respond as promptly to a load change as can the Francis type, especially one of moderate specific speed. Therefore, a propeller unit requires a relatively larger WR^2 of revolving masses to assure the same stability.

General rules may be stated representing present-day good practice Such rules serve as a guide for preliminary design. Each individual plant, however, has particular characteristics that make it differ from some similar plant previously built. The final decision, therefore, should be the result of considerable study as well as of consultation with experienced manufacturers and turbine designers [3, 4, and 5].

In selecting a turbine an effort is made to use a type of runner which will give the required power at the greatest practicable speed, for this reduces the size and cost of hydraulic and electrical equipment, and powerhouse. There are practical limits, however, for any given set of conditions which restrict the speed that can be used and therefore more or less determine the type of wheel to be installed The type of wheel is usually determined by the specific speed.

5. Specific Speed. The specific speed provides a means of comparing the speed of all types of hydraulic turbines on the same basis of head and horsepower capacity. A single runner having a higher specific speed than another therefore runs at a higher number of revolutions per minute to deliver the same horsepower under the same head. By definition, specific speed is the number of revolutions per minute at which a given runner would revolve if it were so reduced in proportions that it would develop 1 hp under 1 ft head. All homologous wheels of the same type, but of different size, have the same specific speed.

The equation for calculating the specific speed for a given wheel from data obtained by test is as follows:

$$N_s = \frac{N\sqrt{P}}{H^{5/4}} \tag{1}$$

where N_s = specific speed;
N = revolutions per minute at full gate and head H;
P = horsepower per runner;
H = head in feet.

Experience has determined that there is a range of heads and specific speeds for which each type of turbine is suitable. Special conditions may sometimes dictate departure from the common practice thus indicated. The tabulation shows the range of speed and heads at which the various types of wheels are commonly used.

TYPE	HEAD, FT	N_s
Impulse	800 and up	(7) 5.5 to 3 (per one jet)
Francis	50 to 800	(110) 80 to 22
Propeller	15 to 100	(200) 170 to 85

An empirical equation which may be used to determine the desirable specific speed for Francis-type wheels is

$$N_s = \frac{5050}{H + 32} + 19 \qquad [2]$$

where H is head in feet.

A similar equation for propeller wheels is

$$N_s = \frac{7000}{H + 32} + 35 \qquad [3]$$

These equations are based on a study of a number of successful installations and though they cannot be considered absolute in general they represent conservative practice for "normal" settings. (See also Section 7.)

There is quite a gap between the specific speed obtained with an impulse wheel and that with a Francis turbine, on the basis of the same head and output. Even a four-jet arrangement will bring the specific speed to only 11 for 800-ft head, against one of 22 for a Francis type. In certain cases it is necessary to design a Francis type of lower than 22 specific speed. Fairly satisfactory Francis-type units of as low as 13.5 specific speed are in commercial use where it was necessary to match the revolutions per minute of an existing generator (Olmstead plant, Provo, Utah, head 330 ft, capacity 3600 hp, speed 300, N_s - 12.8).

Figure 15 shows curves of specific speed against head based on the above equations which may be used in preliminary studies to determine the approximate specific speed and the type of wheel to be used.

The adjustable-blade propeller type gives about 20% more power than the fixed-propeller type for the same head and diameter. For the same power capacity, therefore, about 10% less diameter is required and the speed is about 10% faster than for the fixed-blade type. Therefore, in using Fig. 15, the specific speed of the adjustable-blade unit may be assumed as about 10% higher than that of the corresponding fixed-blade wheel.

Usual specific speed (N_s) for impulse wheels for various heads is given in Fig. 15. Equations for the specific speed of impulse wheels for one and two jets, when the diameter of jet and pitch circle are known, are given in Section 12.

It should be pointed out that, as a practical matter, there is considerable latitude in the specific speed of runners which can advisedly be used for

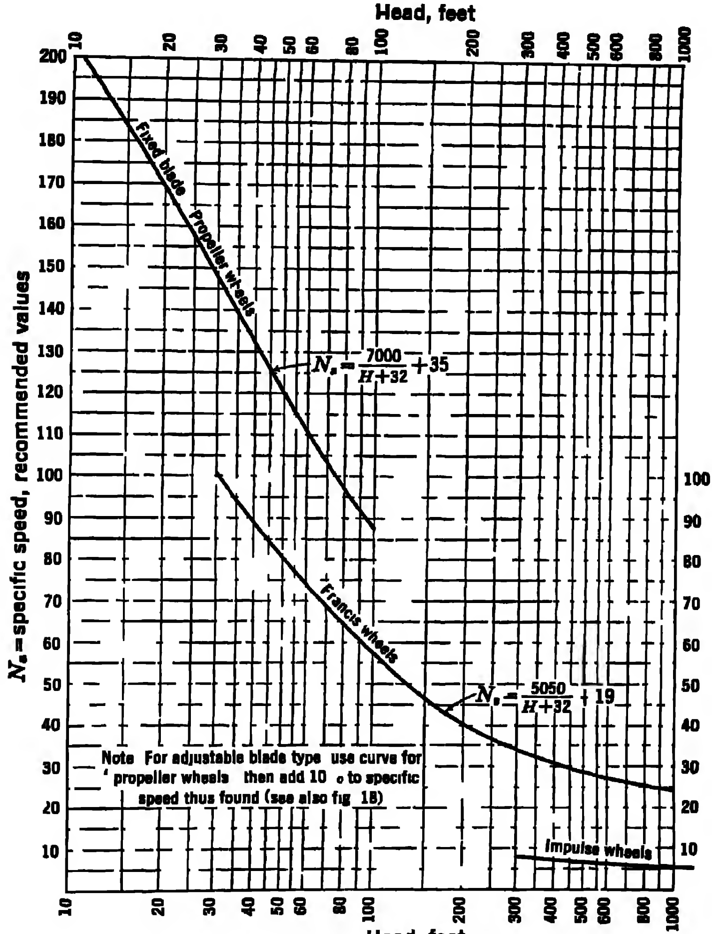


FIG 15 Approximate recommended specific speed for hydraulic turbines

given conditions of head and power, provided that the elevation of the runner, in reference to tailwater, is such as to avoid danger from serious cavitation is discussed in Section 7 The relationship between specific speed (N_s), head, and setting in relation to tailwater (H_s) is shown in Fig 18

To transfer specific speed given in metric units to the English system, the following equation may be used:

$$N_s(\text{English}) = N_s(\text{metric}) \times 4.45 \qquad [4]$$

6. Short-cut Determination of Speed and Diameter. In preliminary studies, knowledge of the approximate cost of the turbine and generator as well as that of other equipment for the plant is frequently desired. Speed is an important determining factor of cost, particularly for the generator. The suitable specific speed having been found from Fig. 15, the use of Fig. 16 for Francis runners will be found convenient in such preliminary studies. If the specific speed (N_s), the capacity, and the head of the desired unit are known, the approximate revolutions per minute may be quickly found, or if head, horsepower, and revolutions per minute are known, the approximate specific speed (N_s) may be found.

Thus, assume that the desired specific speed (N_s) is 70, the horsepower 20,000, and the head 70 ft. Beginning at the top of Fig. 16 at $N_s = 70$, follow down a vertical line until it intersects the slanting head line for $H = 70$ ft; then from this intersection follow across a horizontal line until it intersects the slanting line for hp = 20,000, and read at the bottom of the figure the resultant speed, which is 100 rpm. If the speed read from the diagram is not a synchronous speed, determine the nearest synchronous speed by means of Eq. 17.

The diameter of the runner contributes to the determination of the spacing of the units and is thus one of the most important elements in determining the cost of substructure, superstructure, and width of tailrace.

For such preliminary studies, Fig. 17, giving the approximate runner discharge diameter of Francis units for various heads and capacities of units, will be helpful. Though this diagram does not indicate the specific speeds for the various heads, the runner diameters have been so selected that they correspond to the average type of runner which is used for the particular conditions of head.

Once having the runner discharge diameter, the engineer may determine the principal dimensions of the power plant by the use of Fig. 29, applying to vertical-shaft spiral-case setting.

Figures 16 and 17 should be used only as a short cut or first approach in determining speed and diameter in preliminary computations, but, in general, results obtained will be on the side of conservatism Even in preliminary studies, it will also be found desirable to determine approximate speed and diameter and setting by the more accurate methods outlined below.

7. Cavitation. The use of a runner having a specific speed greater than the recommended values (see Fig. 15) may result in cavitation. This may be avoided by setting the runner at a lower elevation, resulting in increased excavation [5].

Cavitation occurs when the vacuum normally found at local points within the runner and draft tube reaches a value at which bubbles of water vapor are formed. These bubbles suddenly collapse with a violent action farther along the path when the pressure is slightly increased. The pressure at which vapor is formed is dependent on the atmospheric pressure and water temperatures.

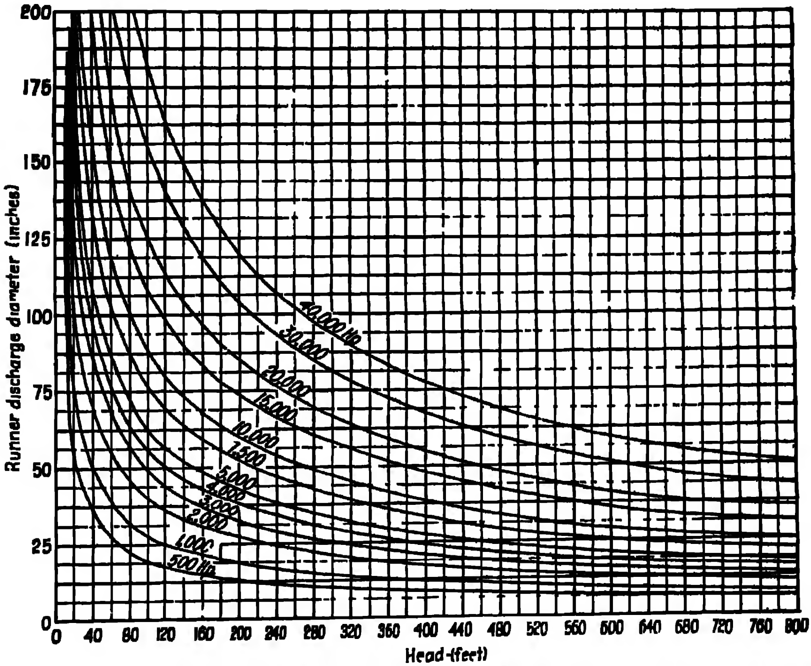
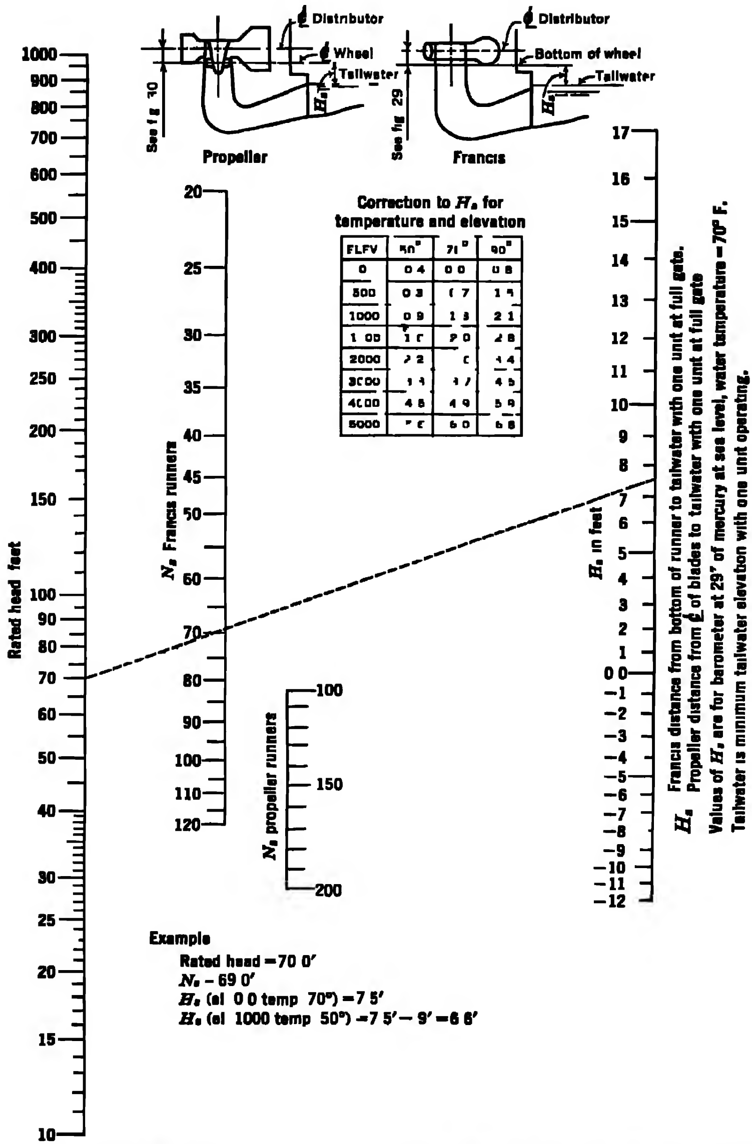


FIG. 17 Discharge diameters of Francis runners required to develop various capacities under different heads. Based on normal specific speed

Extreme cavitation causes a loss of efficiency, but continued action confined to local spots may result only in pitting of the metal and concrete surfaces where the bubbles of vapor collapse, breaking down the surface so that the clean metal exposed oxidizes rapidly and eventually produces a cavity. By this continued process, honeycombing of the material of the runner or draft tube results. Pitting may also occur from other causes such as the action of acid waters.

Where cavitation is bad, runner blades having a thickness of 1 in. may be corroded through in less than a year's operation. The usual method of repair is to weld on new material as described in Section 23, Chapter 44. Porous materials like cast iron are least resistive to damage, whereas materials of a dense, homogeneous structure are better suited to resist cavitation. Some manufacturers cast a depression in the surfaces particularly subject to



Correction to H_s for temperature and elevation

ELEV	50°	70°	90°
0	0 4	0 0	0 8
500	0 3	[illegible] 7	1 [illegible]
1000	0 9	1 3	2 1
1 [illegible]00	1 [illegible]	[illegible] 0	[illegible] 8
2000	[illegible] 2	[illegible]	[illegible] 4
3000	[illegible]	[illegible]	4 5
4000	4 5	4 9	5 9
5000	[illegible]	6 0	6 8

FIG 18 Chart showing recommended height of setting for hydraulic turbines for preliminary studies only

cavitation damage and weld in stainless steel. Damage to the runner usually takes place on the back of the bucket or blade near where the water leaves the wheel.

High velocities with sudden changes in velocity which result in low-pressure areas are conducive to cavitation. This can be avoided either by improved design which eliminates sudden changes in curvature and/or by lowering the setting of the runner with reference to tailwater, so that a vacuum sufficient to produce vaporization will not occur. In some cases it is necessary to set the runner below tailwater level. This is particularly true of propeller turbines [5].

In general, the higher the specific speed of the runner for the same power and head, the higher the velocities through the runner and, therefore, the lower must be the setting to avoid cavitation. As the head is increased on a runner of given specific speed, the velocity through the runner becomes higher and the setting must be lowered accordingly to avoid cavitation.

The safe height of setting above or below tailwater (the value H_s in Fig. 18) varies with the head and specific speed and may be determined by tests of model runners in especially equipped cavitation laboratories or by past experience records on existing installations [14].

Since there is considerable variation in the operation of hydroelectric turbine units, it is impossible to state rigid general rules for defining the setting for a given turbine. For a plant with a large number of units, it is possible that one unit will be operated alone only a relatively few hours a year. If all the units are set at an elevation corresponding to the low tailwater (H_s of Fig. 18) obtained with only one unit operating, then the deeper tailrace excavation which is required may be too expensive. Often it is more economical to repair, once a year, by welding, the damage done by cavitation than to prevent all cavitation. (See Chapter 44, Section 23.)

Figure 18 is a chart showing the setting for Francis and propeller runners of different specific speeds at different heads. In general the values given by the chart in Fig. 18 are conservative and are intended to be used for preliminary estimates only. Final value of H_s should be determined by consultation with the turbine manufacturers, and in some cases model cavitation tests may be desirable [14]. Examples of the use of this chart for determining the height of setting are given in Sections 15 and 16.

8. Efficiency of Turbines. Consideration must be given, in selecting the type of wheel for any set of conditions, to the matter of part-load efficiency. The maximum efficiency at normal specific speeds has been developed so that, for runners varying from 20 to 160 in specific speed, a maximum efficiency of 90 to 92% may be expected with a properly designed vertical-shaft setting. For specific speeds up to 200, maximum efficiencies above 88% are common. For impulse wheels the maximum efficiency is usually between 85 and 87%.

The part-load efficiency differs greatly for different specific speeds and types of turbines. The percentage of load at which the maximum efficiency

occurs varies somewhat with the specific speed, although the turbine manufacturer can usually vary this condition considerably to suit the customer's needs. The following table and the curves in Figs. 19, 20, and 21 show the variation in part-load efficiencies with different types of wheels.

TABLE 2

TYPICAL PART-LOAD EFFICIENCY OF TURBINES FOR VARIOUS SPECIFIC SPEEDS
Values taken from Figs. 19, 20, and 21

Type	Specific Speed	Per Cent Efficiency: $\frac{1}{4}$ Load	$\frac{1}{2}$ Load	$\frac{3}{4}$ Load	Full Load	Maximum	Per Cent Load for Maximum Efficiency
Impulse	5	81	86	87	85	87.1	70
Francis	17	62	83	88	83	88	75
Francis	25	60	85	90	84	90.2	80
Francis	50	59	83	90	85	91.5	85
Francis	75	54	82	91	86	91.0	87.5
Francis	92	47	71.5	85	87	91.5	92.5
Francis	103	55	74.5	86.5	86	92.5	92
Fixed-blade propeller	155	45	70	84.5	82	91.5	92
Fixed-blade propeller	180	32	59	78	84	88	96
Adjustable-blade propeller	169	83.5	91	91 5	87	91.6	70

Except for the Kaplan turbine, it is seen that, as the specific speed increases, the average part-load efficiency decreases. Figure 21 shows a comparison of the part-load efficiency of the Kaplan, propeller, and a very high-speed Francis runner.

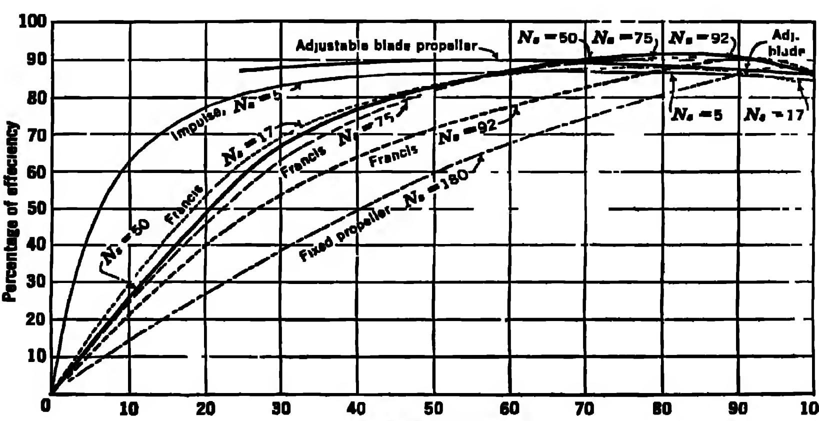


FIG. 19. Efficiency of various types of hydraulic turbines.

The part-load efficiency characteristics may affect the choice of the type of wheel to be used. For a plant with a variable load the advantages of the higher specific speed wheel may be offset by lower part-load efficiencies.

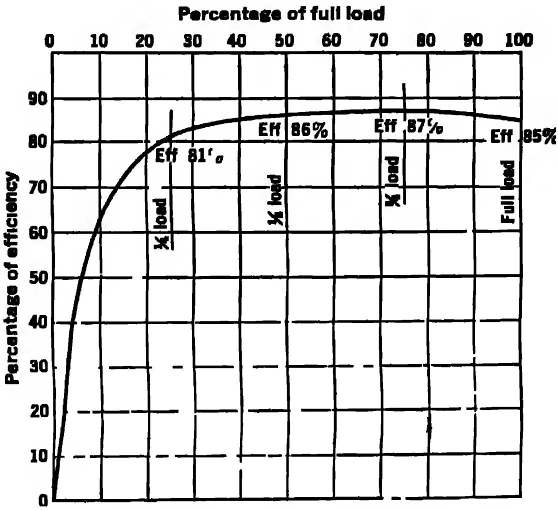


FIG. 20 Efficiency curve for impulse wheel $N_s = 5$

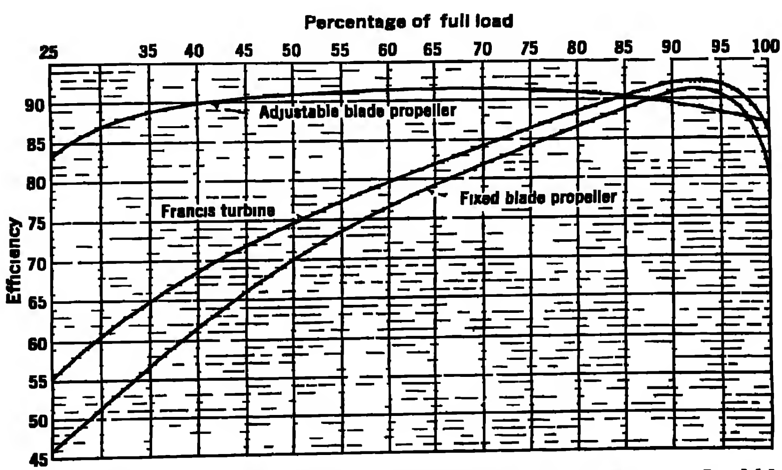


FIG. 21 Efficiency curves of adjustable-blade propeller, 164 rpm; fixed-blade propeller 150 rpm, and Francis turbine 100 rpm, 38-ft head, 9500 hp (S Morgan Smith)

9. Number and Size of Units. In determining the number and size of units for any given development, due consideration should be given to available water supply, its distribution throughout the year, pondage available,

and the size and characteristics of the connected-load curve [3]. (See also Chapter 10, Section 4.)

All other things being equal, the larger the size of the individual units, the lower will be the capital cost and the cost of maintenance per kilowatt. It is obvious that it would not be desirable to have a one-unit plant if this unit were the sole source of power for the connected load, nor would it be desirable to install in a run-of-river plant a unit so large in relation to the available water supply that it would have to operate at low gate opening a material part of the time. Also, a unit should not be larger in capacity than the minimum load of the system to which it is connected.

The number of units may be affected by the part-load characteristics of the turbines selected. For a plant with a variable load, if the wheels have a relatively poor part-load efficiency, there is more advantage in increasing the number of units and decreasing their size, for then the lighter plant loads become a greater percentage of the unit capacity and the plant efficiency is thereby increased, since some of the units will be shut down at the light plant loads. This gain in efficiency is offset by an increased cost per installed horsepower with smaller units. The final choice involves an economic study in which the value of power, fixed charges on the units, and other factors must be weighed [15].

In a plant using propeller wheels with a variable load, the combination of some fixed-blade units and some adjustable-blade units can be operated so that at the very small plant loads only the Kaplan units will be running, producing the power at higher efficiency than if all units were of the fixed-blade type [22].

10. Homologous Equations. The characteristics of each type or design of turbine are usually determined by the manufacturer by a careful test of a homologous model. The characteristics of the full-size runner can then be predicted within engineering accuracy by means of the following homologous equations, provided that the setting, gate angle, and velocity ratio are the same for model and prototype.

The velocity ratio, sometimes known as ϕ (phi), the peripheral coefficient or speed factor, is the ratio of the peripheral velocity of the buckets at the nominal inlet diameter to the spouting velocity of water under the head acting on the turbine:

$$\phi = \frac{\pi(D_1/12)(N/60)}{\sqrt{2gh}} \qquad (5)$$

where D_1 = inlet diameter of runner in inches;
N = revolutions per minute;
ϕ = peripheral coefficient;
g = acceleration of gravity = 32.2 ft per second per second;
h = head in feet.

A curve of ϕ at best speed and full gate plotted against N_s is shown in Fig. 22. The peripheral coefficient should not exceed the value given in Fig. 22. Thus,

if we consider a Francis runner with a discharge diameter of 120 and an N_s of 40, we see from Fig. 22 that the inlet diameter should be 102½% of the discharge diameter, or 123 in.

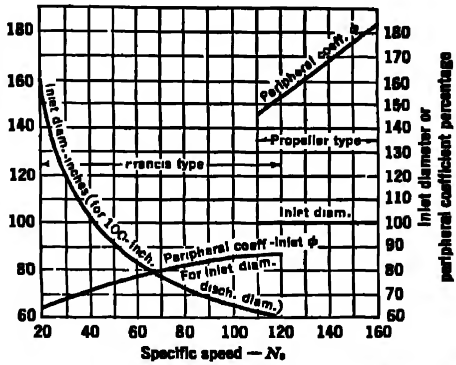


FIG. 22. Inlet diameter and peripheral coefficient for 100-in. discharge diameter. Thus, if the specific speed of a Francis runner is 65 and its discharge diameter is 130 in., its inlet diameter should be $0.80 \times 130 = 124$ in.

Knowing the efficiency, speed, power, and discharge of a runner of a given diameter at a given head, we may calculate directly from the following equations the power, speed, and discharge of a homologous runner of a different diameter, under a different head, for the same efficiency.

For constant diameter,

$$\frac{P_2}{P_1} = \left(\frac{H_2}{H_1}\right)^{3/2} \qquad [6]$$

$$\frac{N_2}{N_1} = \left(\frac{H_2}{H_1}\right)^{1/2} \qquad [7]$$

$$\frac{Q_2}{Q_1} = \left(\frac{H_2}{H_1}\right)^{1/2} \qquad [8]$$

For constant head,

$$\frac{P_2}{P_1} = \left(\frac{d_2}{d_1}\right)^2 \qquad [9]$$

$$\frac{N_2}{N_1} = \frac{d_1}{d_2} \qquad [10]$$

$$\frac{Q_2}{Q_1} = \left(\frac{d_2}{d_1}\right)^2 \qquad [11]$$

where P_1 and P_2 = horsepower for different conditions;
d_1 and d_2 = runner diameter, in inches, for different conditions;
H_1 and H_2 = head, in feet, for different conditions;
Q_1 and Q_2 = discharge, in cubic feet per second, for different conditions;
N_1 and N_2 = revolutions per minute for different conditions.

These six equations may be summarized as follows:

The horsepower of a runner changes as the square of its diameter, as the three-halves power of the head, or directly as the discharge.

The revolutions per minute of a wheel change in proportion to the square root of the head and inversely in proportion to the diameter of the runners.

The discharge from a wheel will vary in direct proportion to the power, as the square of the runner diameters, and as the square root of the head.

For example, let a known runner have the following characteristics:

$$P_1 = 1000 \text{ hp}$$
$$d_1 = 50 \text{ in.}$$
$$H_1 = 100 \text{ ft}$$
$$N_1 = 180 \text{ rpm}$$
$$Q_1 = 100 \text{ cu ft per second}$$

Let it be desired to know the required diameter, speed, and discharge of a runner to operate at 150-ft head and give 2000 hp at the same efficiency.

From Eqs. 6 to 8 the characteristics of the known runner under 150 ft head would be

$$P_2 = 1000(\tfrac{150}{100})^{3/2} = 1837 \text{ hp}$$

$$N_2 = 180(\tfrac{150}{100})^{1/2} = 220 \text{ rpm}$$

$$Q_2 = 100(\tfrac{150}{100})^{1/2} = 122.5 \text{ cu ft per second}$$

To give 2000 hp under this head of 150 ft, the turbine must have the following characteristics, from Eqs. 9 to 11:

$$d_2 = 50(\tfrac{2000}{1837})^{1/2} = 52.2 \text{ in.}$$

$$N_2 = 220\,\frac{50.0}{52.2} = 210 \text{ rpm}$$

$$Q_2 = 122.5\left(\frac{52.2}{50.0}\right)^2 = 133.4 \text{ cu ft per second}$$

11. Diameter and Speed of Reaction Turbines. When the specific speed and power per unit have been selected for the given conditions of head and plant discharge, it is possible to determine the physical size of the turbine. To a large extent the diameter of the turbine determines the dimensions of both the substructure and superstructure of the power plant.

The nominal diameter, the size by which a given wheel is known, is different for different types of wheels. For Francis runners the nominal diameter is taken by some manufacturers as the average inlet diameter and by others as the discharge diameter. In order to be consistent, the nominal diameter is taken as the discharge diameter in this book. The discharge diameter is measured at the bottom where the water leaves the runner, as indicated in Fig. 3. The relation between inlet and discharge diameters for Francis runners is given

of the theoretical velocity under the net head acting on the nozzle. From this it can be stated that, for the best speed of an impulse wheel,

$$N = \frac{\sqrt{2gH} \times 0.46 \times 60}{3.1416 \times D/12} = \frac{845(H)^{1/2}}{D} \qquad [12]$$

where N = best speed, revolutions per minute;
H = head on nozzle, feet;
D = pitch diameter of bucket circle, inches.

The equation for the full-load horsepower of an impulse wheel with a single power jet is as follows, assuming a full-load efficiency of 85%.

$$P = \frac{0.85 \times 0.7854 \times d^2 \times 62.5 \times \sqrt{2gH} \times H}{144 \times 550} = \frac{d^2H^{3/2}}{240} \qquad [13]$$

from which diameter in terms of power

$$d = \frac{15.5\sqrt{P}}{H^{3/4}} \qquad [14]$$

where d = diameter of free jet, inches;
H = head on nozzle, feet.

If two power jets of diameter d are used and so disposed that they do not interfere, the power is twice the value given by the above equation. Strictly, it is a trifle more because windage and friction remain as before. The best speed with two jets is practically the same as with one jet.

The diameter of the nozzle opening at the tip is usually from 10 to 20% greater than the free jet diameter, depending on the design of the nozzle and needle tip.

From the above two equations for speed and power it is possible to show that the specific speed of a single jet impulse wheel

$$N_s = \frac{845\sqrt{H}}{D} \times \frac{dH^{3/4}}{15.5} \times \frac{1}{H^{5/4}} = 54.5\frac{d}{D} \begin{cases} \text{for efficiency} & = 85\% \\ \text{for } \phi & = 0.46 \\ \text{for jet velocity } v & = 0.985\sqrt{2gh} \end{cases} \qquad [15]$$

where d and D are the diameters, in inches, for the free jet and pitch circle of the runner, respectively. Impulse wheels with low specific speed (N_s of 4 to 5) give the best efficiencies. On the other hand, higher speed means lower capital cost.

For an impulse wheel with Z jets, the specific speed is derived as follows:

$$N_s' = N_s\sqrt{Z} \qquad [16]$$

N_s applies to one jet.

For Eq. 15, if efficiency is different from the 85% assumed, multiply by ratio of efficiencies.

For practical reasons the diameters of jets commonly used are not greater than 8 in., although a jet 14 in. in diameter has been successfully used.* If it is desired to transfer specific speed English system to specific speed metric system, or vice versa, Eq. 4, Section 5, may be used.

With the wheel and jet diameters D and d obtained as described above, it is desirable to determine the diameter of nozzle pipe (and shutoff valve connected thereto), width of wheel pit, and size of buckets.

The diameter of the nozzle pipe should be 3.5 to 4 times the jet diameter d. The velocity through the nozzle-pipe inlet should be limited to 0.1 $\sqrt{2gh}$ for low heads and not over 28 to 30 ft per second for high heads. The width of the wheel pit should be about 10 times the jet diameter d if one jet is used per wheel and correspondingly greater if two jets are employed, depending on how they are disposed on the wheel.

13. Synchronous Speed. Since modern hydraulic turbines are usually directly connected to alternating-current generators which must run at synchronous speed, the best speed at which the turbine is designed to run should correspond with the generator synchronous speed as closely as possible. The generator must have an even number of pole pieces, each of which may be considered as producing a half cycle. The synchronous speed for the generator is

$$N = \frac{120f}{p} \qquad [17]$$

where p = the number of pole pieces and must be an even whole number;
N = revolutions per minute;
f = the cycles per second.

It is not usually possible to match a synchronous speed exactly with the best speed of the turbine. Ordinarily, if the speed of operation is within 5% of the best speed of the runner, no appreciable decrease in efficiency occurs. The allowable variation in speed varies with the type of wheel used. It might also be noted here that a similar effect can be obtained with great fluctuation in head, and where large head variations are encountered a study of the effect on the efficiency of the runner selected should be made.

To vary the cycles with an existing generator of f poles (Eq. 17) the revolutions per minute (N) of the prime mover must be changed accordingly. The exchange of energy from one system of one fixed number of cycles to one of another number of cycles requires a corresponding speed change of prime mover at the plant.

The southern part of California, for instance, still has an extended distributing system at 50 cycles, but also one of 60 cycles. Blocks of hydro energy are frequently delivered to one system from the other system. Thus units at the United States Hoover plant and several units of plants at Big Creek, California, of the Southern California Edison Company, Los Angeles,

* See Table 1, San Francisquito No. 1 Plant.

and also some units of the Bureau of Power and Light, Los Angeles, San Francisquito No. 1 Plant, and Big Creek Plants No. 1 and No. 2A may be operated at either 50 or 60 cycles.

With impulse-wheel units, the change from one cycle speed to the other can be done without appreciable sacrifice in efficiency, thanks to the flexibility of design of buckets and to the fact that operation is as smooth at one cycle speed as at the other.

Francis turbines, though not showing serious sacrifice as to either efficiency or output, do not operate with equal smoothness at both cycle speeds.

14. Example of Selection of Runner for Low Head. The following example is given to illustrate the method in which the graphs and equations in the preceding sections may be used to determine preliminary dimensions of turbines so that other parts of the power plant may be proportioned to obtain a preliminary design and estimate.

Suppose the net head on a proposed plant is 70 ft, the discharge capacity per unit 600 cu ft per second, expected full-gate efficiency 88%, and the unit is to be connected to a 60-cycle generator. Determine the specific speed, diameter, and revolutions per minute suitable for the turbine.

Solution:

$$\text{horsepower per unit} = \frac{62.4}{550} \times \text{Head} \times \text{Discharge} \times \text{Efficiency} =$$

$$\frac{62.4 \times 70 \times 600 \times 0.88}{550} = 4200 \text{ hp}$$

Specific Speed. Figure 15 shows that the recommended specific speed (N_s) at 70-ft head for a Francis wheel is 69.

Use the curve for horsepower at 1-ft head for a 100-in.-diameter Francis wheel in Fig. 23. It is found that horsepower for 1-ft head is 14.7 for a Francis runner having an N_s of 69.

Discharge Diameter. The above horsepower is for a 100-in.-diameter runner at 1-ft head. The given head is 70 ft. Therefore, using Eq. 6, Section 10, the horsepower of a 100-in.-diameter wheel at 70-ft head will, for Francis unit N_s, be 69.

$$\frac{P_2}{P_1} = \left(\frac{H_2}{H_1}\right)^{3/2} = \frac{P_2}{14.7} = \left(\frac{70}{1}\right)^{3/2}$$

$$P_2 = 70\sqrt{70} \times 14.7 = 8620 \text{ hp for 100-in. runner}$$

But the desired power of the wheel is 4200 hp; accordingly, to find the required discharge diameter use Eq. 9, Section 10:

$$\frac{P_2}{P_1} = \left(\frac{d_2}{d_1}\right)^2 = \frac{4200}{8620} = \left(\frac{d_2}{100}\right)^2$$

from which $d_2^2 = 0.487 \times 10{,}000$

$d_2 = 70$ in.

which is the required discharge diameter of a Francis runner having a capacity of 4200 hp at 70-ft head.

Revolutions per Minute. As yet, it is not known at what number of revolutions per minute the turbine will revolve. Using Fig. 23, it is found that the revolutions per minute for a 100-in. Francis runner at 1-ft head, having an N_s of 69, will be 18.2.

But the head is to be 70 ft. Using Eq. 7 of Section 10,

$$\frac{N_2}{N_1} = \left(\frac{H_2}{H_1}\right)^{1/2} \frac{N_2}{18.2} = \sqrt{\frac{70}{1}}$$

$$N_2 = 18.2\sqrt{70} = 152.8$$

which is the revolutions per minute of a 100-in.-diameter Francis wheel having an N_s of 69 at 70-ft head.

But it is already known from the above that the diameter of the 4200-hp Francis runner at 70-ft head is 70 in. Therefore, use Eq. 10 of Section 10,

$$\frac{N_2}{N_1} = \frac{d_1}{d_2} \frac{N_2}{152.8} = \frac{100}{70}$$

$$N_2 = \frac{100 \times 152.8}{70} = 218 \text{ rpm}$$

N_2 may also be found from Eq. 1, whence

$$N_2 = \frac{69 \times 70^{5/4}}{\sqrt{4200}} = 216$$

The speed may also be found from Fig. 16. Using Eq. 17, Section 13 (for 60 cycles),

$$N = \frac{7200}{P}$$

$$218 = \frac{7200}{P}$$

we find that P, the number of poles, would be 33, which is not an even whole number and therefore 218 would not be a synchronous speed. If we use 32 poles, the revolutions per minute for 60 cycles speed would be $N = 7200/32 = 225$ rpm, which is near enough to be satisfactory for the purpose.

Summary of Findings. For a 4200-hp Francis runner to be connected to a 60-cycle generator at 70-ft head, it has been determined that

specific speed $N_s = 69$
discharge diameter $= 70$ in.
speed $= 225$ rpm

For determining these characteristics for a fixed-blade propeller or for an adjustable-blade propeller, the procedure would be similar, using the proper curves of the same Fig. 23.

15. Example of Height of Setting of Runner. Assume that the plant in the preceding example is located within 100 ft of sea level, where the lowest barometer reading will be about 29 in. of mercury, and the water temperature about 70 degrees Fahrenheit maximum. The recommended height of setting H_s, which is the vertical distance from low tailwater to the bottom of the Francis runner, is obtained from the chart on Fig. 18 as follows: The rated head is 70 ft, the value of N_s is 69. A straightedge placed so that one end rests on 70 on the head scale at the left and crosses the specific-speed line at 69 intersects the H_s line on the right at about 7.5 ft. Thus the recommended height of setting for the bottom of the Francis runner is 7.5 ft. The approximate distance from the bottom of the runner to the center line of the distributor is given on Fig. 29 as $0.37D$ or $0.37 \times 70/12 = 2.2$ ft.

The center line of the distributor is thus found to be $7.5 + 2.2 = 9.7$ ft above low tailwater for the Francis runner.

A higher-speed Francis runner may be used by lowering the setting. Figure 18 shows that a Francis runner with a specific speed of 80 may be used at 70-ft head provided that the distance from the bottom of the runner to low-tailwater level is made 0.5 ft.

Going through the same procedure as in the example given in Section 14, but with $N_s = 80$, it is found that, for a 4200-hp Francis wheel at 70-ft head, the discharge diameter would be 65.5 in. and the revolutions per minute would be 257 Manifestly, there would be some saving in the cost of generator and turbine over the cost for the 70-in.-diameter wheel at 225 rpm. On the other hand the required low setting 0.5 ft above minimum low tailwater might make the total cost greater than for the 70-in. wheel. This is just one of the factors that must be considered in selecting a turbine.

If the tailrace elevation of the plant was 500 ft above sea level and the water temperature reached a maximum of 90° F, the table on Fig. 18 shows that the setting of the runners would be 1.5 ft lower than when the plant is located near sea level with a maximum water temperature of 70° F.

The values given by the chart in Fig. 18 represent average safe conditions and are intended to be used for preliminary investigations. A turbine manufacturer may suggest changing the value of H_s several feet to suit the conditions for his particular runner.

The above example is based upon the assumption that the net head of 70 ft remains practically constant during the season. If this is not so, a runner should be selected of such characteristics that the total kilowatt-hours available from head and water quantities during the year are the maximum. Assuming, for instance, that the unit draws from a storage reservoir so that the available head decreases toward the end of the season, then a runner with a lower speed and a greater flexibility in efficiencies should be considered.

16. Example of Selėction of Runner for High Head. The head is 700 ft (assumed to remain constant); the total discharge capacity of one unit is 200 cu ft per second. Determine the type, size of wheel, and speed.

For this illustrative example, both the impulse and Francis types will be considered and compared.

The impulse wheel must be set above the level of high tailwater to avoid drowning the jet.

Reference to Fig. 20 shows that the maximum efficiency to be expected is 87% and full-load efficiency is about 85%. The horsepower capacity of the impulse unit is then

$$\frac{62.4 \times 700 \times 200 \times .85}{550} = 13{,}500 \text{ hp}$$

Specific Speed and Type. Figure 15 shows that the recommended specific speed at 700-ft head is 6 for an impulse wheel.

If a single runner and nozzle is used, the horsepower per runner is 13,500. If a double overhung unit having two runners, each having one nozzle, is used, the horsepower per runner is 6750.

Diameter of Jets. From Eq. 14, Section 12, the diameter of the jet may be found as follows:

$$d = \frac{15.5\sqrt{P}}{H^{3/4}} = 15.5\frac{\sqrt{13{,}500}}{(700)^{3/4}} = 13.25 \text{ in. for single runner}$$

Using the same formula, but with $P = 6750$ for a double overhung unit, the diameter of each jet (one jet per wheel) = 9.37 in. for double runner.

Pitch Diameter of Runner. From Eq. 15, Section 12, the diameter of the pitch circle of the wheel is

$$D = \frac{54.5d}{N_s} = \frac{54.5 \times 13.25}{6} = 120 \text{ in. for single runner}$$

Similarly, for the double unit,

$$D = \frac{54.5 \times 9.37}{6} = \text{pitch diameter} = 85 \text{ in. diameter}$$

Speed. The best speed, as found from Eq. 12, Section 12, would be

$$N = \frac{845\sqrt{H}}{D} = \frac{845\sqrt{700}}{120} = 187 \text{ rpm for single runner}$$

Similarly,

$$N = \frac{845\sqrt{700}}{85} = 263 \text{ rpm for double runner}$$

Synchronous speed for 60 cycles (Eq. 17, Section 13)

$$p = \frac{7200}{N} = \frac{7200}{187} = 38.5 \text{ (use 38 pole pieces, single runner)*}$$

$$N = \tfrac{7200}{38} = 189.5 \text{ rpm for single runner}$$

Similarly

$$p = \tfrac{7200}{263} = 27.4 \text{ (use 28 pole pieces, double runner)}$$

$$N = \tfrac{7200}{28} = 257 \text{ rpm for double-runner unit}$$

Summary. The data for a Francis-type wheel, worked out in the same manner as the example in Section 14, are given for comparison with the impulse units. It happens that, if the maximum recommended specific speed of 26 for 700-ft head is used, the best speed becomes 768 rpm. Hydraulic turbines, especially in vertical setting, are usually not run at speeds much greater than 600 rpm in the larger sizes, so that the data for a specific speed of 20 were used for this example. For probable efficiency see Fig. 19.

	Francis	Single-Impulse Runner	Double-Impulse Runner
Specific speed	20	6	6 per jet
Jet diameter	. .	13.25 in.	9.37 in.
Wheel diameter	40 in. discharge 63 in. inlet	120 in. pitch	85 in. pitch
Best speed	615 rpm	187 rpm	263 rpm
Synchronous speed	600 rpm	189.5 rpm	257 rpm
Number of pole pieces	12	38	28
Head utilized	700	700	700
Maximum efficiency	90	87	87
Full-load efficiency	85	85	85
Full-load horsepower	13,500	13,500	13,500

Examination of the data in the above summary shows an advantage as far as speed is concerned in favor of the Francis wheel. This advantage may be offset, however, by other items. It is very likely that the size (total cubic feet) of the powerhouse will be greater for the Francis runner than for the impulse wheels, since the water passages for such a high-speed unit will probably determine the unit spacing. The substructure will be deeper for the Francis runner, although reference to Fig. 18 shows that the bottom of the wheel may be set 11.5 ft above tailwater. The powerhouse superstructure will be higher for the Francis unit if this wheel runs on a vertical shaft. There is no advantage in efficiency over the impulse wheels, since at high heads the sealing rings soon become worn and leakage losses result. One advantage of the Francis unit is that, on a strictly comparable basis, it would have a slightly

* A 38-pole generator not being standard may cost more than a 36- or 40-pole. Therefore, the speed may be chosen accordingly.

greater effective head (perhaps 5 ft) because with the impulse wheel the head from the center line of the jet to the tailwater is discarded. In the example this is not considered. If the difference were 5 ft, it would amount to only about 0.7% of the 700 ft.

This question of Francis versus impulse wheels at high head, particularly with regard to efficiency, is one on which not all engineers are in agreement. If sealing rings are tight as when the unit is first placed in service, the efficiency of the Francis unit is usually considerably higher than that of a corresponding impulse wheel. Thus, at the Nantahala development of the Aluminum Company of America in North Carolina, a Francis unit (Newport News, 1942) gave a maximum efficiency of 93.7% on test. This was a 60,000-hp unit under 925-ft head, 450 rpm, specific speed 21.6.

The single-runner impulse wheel is the lowest in speed and also requires an exceptionally large jet, calling for heavy nozzle equipment. The double overhung runner operates at about 50% higher speed and requires nozzles of more common size. The final selection should be based on a complete preliminary estimate for each of the three cases.

If the Francis turbine could be operated at all times at its best-efficiency gate opening, it would show a decided advantage over the impulse wheel. Comparison of efficiency curves plotted in Fig. 19 for an impulse wheel of $N_s = 5$ and a high-head Francis turbine of $N_s = 17$ shows the great advantage of the impulse wheel at loads below 40% output per jet. With the double overhung type, one side can be shut down watertight and the remaining runner operated at its best efficiency

To the practical engineer, the comparison, from a point of view of reliability, accessibility, and repair cost (total expense of outage), is a paramount issue. With a Francis turbine, especially of the vertical-shaft type, the parts most subject to wear are the least accessible. Running clearances must be closely watched and the unit shut down quickly before serious damage is done. The impulse wheel has neither disadvantage Running clearances are only those in shaft and bearings and readily accessible. Parts subject to natural hydraulic wear are the throat ring and the needle tip, which can be replaced in less than an hour's time and at very small expense. Almost every essential part of the entire unit (wheel and generator) can be examined and removed without disturbing its neighbor part. The unit requires no thrust bearing and no auxiliary oiling systems such as are involved in a vertical-shaft high-head Francis-type unit.

17. Runaway Speed. In selecting the ratings and capacities of hydroelectric units, consideration must be given to the relation between the characteristics of the generator and those of the hydraulic turbine.

The generator must be designed to stand the full runaway speed of the turbine to which it is to be connected, under the maximum head conditions at which the unit may operate. The ordinary type of reaction turbine will reach about 180% of its normal speed at runaway; but, if the maximum head on the turbine sometimes reaches 15% above normal, then this same turbine

will reach about 193% of its normal speed at runaway, and the generators should be designed to stand these maximum conditions. For some fixed-blade propeller installations the maximum runaway speed for which the generators are built is as high as 2½ times normal speed.

Figure 24 shows the percentages of runaway speed plotted against percentage normal head. The characteristics of Francis runners having a specific

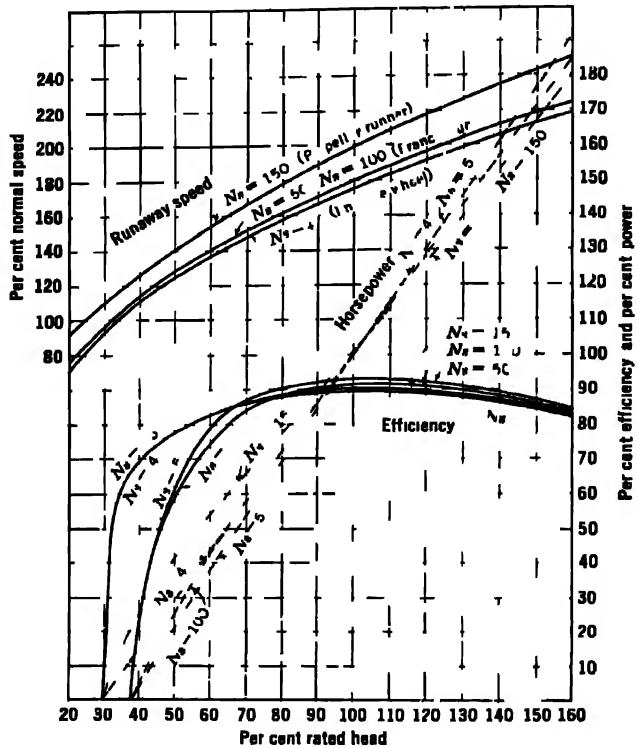


FIG. 24. Efficiency, horsepower, and runaway speed of various specific-speed runners at constant speed under different percentages of rated head.

speed of 50 and of 100 are very similar in this respect. The propeller type of runner of 150 specific speed has a runaway about 20% greater. On this same curve is shown the variation in horsepower of various types of runners under varying head conditions. For instance, the curve shows that a Francis unit with N_s = 50 will deliver at 150% of rated head about 177% of the power which it would deliver at 100% of rated head.

18. Types of Turbine Settings. The "setting of a turbine" is a somewhat vague term, used [illegible] which usually means the casing or flume in which the turbine is set and may also include the draft tube and the distance that the runner is set above tailwater. The setting of the turbine in relation

to tailwater is discussed in Section 7 on cavitation, and draft tubes are considered in Section 20. The impulse wheel requires its own type of setting as described in Section 21. The present section will be confined largely to a discussion of flumes and scroll cases.

The Francis and propeller types require a flume or case, the main function of which is to distribute the water as uniformly and smoothly as possible around the circumference of the guide case in order to obtain maximum power and efficiency. The three general types of setting for these two types of runner are:

1. The open-flume setting as shown in Fig. 25.
2. The steel cylinder case, used only for very small units.
3. The scroll or spiral casing shown in Fig. 26.

By far the most commonly used type of setting for modern installations is the scroll or spiral casing. The various types of scroll cases may be divided into several classes as follows:

1. The concrete scroll case setting as shown in Figs. 32 and 33
2. The plate-steel scroll case setting as shown in Figs 35 and 36.
3. The cast-iron or cast-steel scroll case setting as shown in Fig. 37.

The semi-scroll or heart-shaped scroll as shown in Fig. 32 is used in low-head installations of fairly large capacity in order to obtain a sufficiently low velocity. The ordinary scroll case would require greater unit spacing.

For very low heads a siphon-type setting, as shown in Fig. 31, may be used. Usually, with the semi-scroll type, the top of the casing is above headwater level, the water being drawn up by an ejector at the start and air kept from re-entering by a sealed chamber over the wheel. This allows the wheel to be set higher and decreases the depth of excavation for the draft tube, resulting, in some cases, in a saving in cost.

There is no competitor at very low heads, say up to 30 ft or so, for the open-flume or concrete spiral type of setting. For 60- or 70-ft heads, especially with large units, the matter of reinforcing the concrete casing may be a difficult problem and careful study is required in order to arrange the reinforcing bars in the most advantageous manner. The cost of the reinforcing steel, its placing, and the cost of the rather complicated formwork necessary require careful consideration in comparison with plate-steel casings at this head for large units.

Plate-steel spiral casings are applied over a wide range of heads and may compete with concrete scroll cases for quite low heads, down to, say, 40 ft, and with cast-iron and cast-steel spiral casings up to, say, 400-ft or more head, depending on size.

The fields of application of all these types of casing overlap, depending upon local conditions and size of unit. Figure 27 indicates the approximate limits of head and capacity within which the various types are ordinarily used.

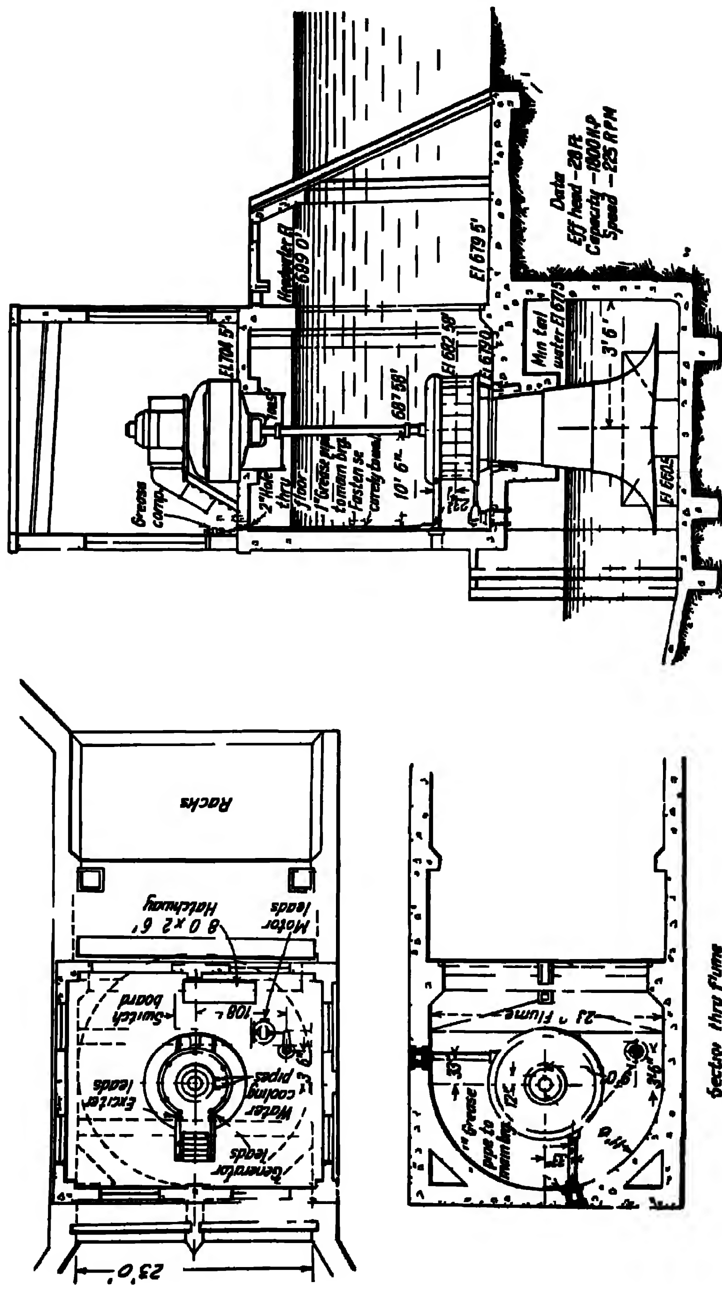


FIG 25 Single-runner vertical open-flume setting Plate-steel draft tube simplifies formwork Gate mechanism located on discharge ring gate shaft extended up to generator floor for connection to governor

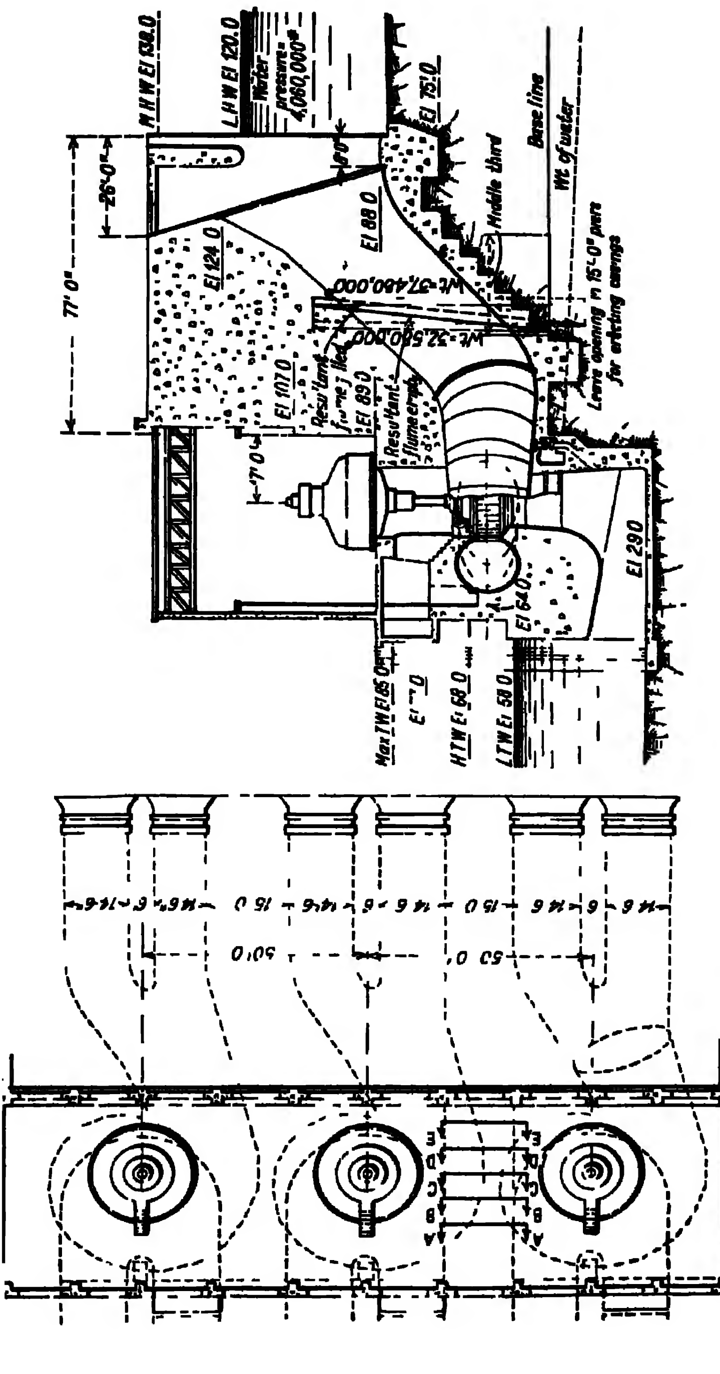


FIG 26 Lower Bonnington plant, British Columbia; plate-steel spiral-cased units, 100 rpm, 70-ft head, 20,000 hp, showing intakes through the dam, elbow-type draft tubes, and special construction of generator air vents because of high floods in tailwater.

19. Flumes and Scroll Cases. Velocity. The matter of water velocity and smoothness of flow at the inlet of the turbine is of great importance and affects the full-load and part-load efficiency of the unit. Figure 28 shows the safe full-load velocities in casings and flumes for the various types of units as determined by experience. Excessive velocities may cause unsteady operation of the unit.

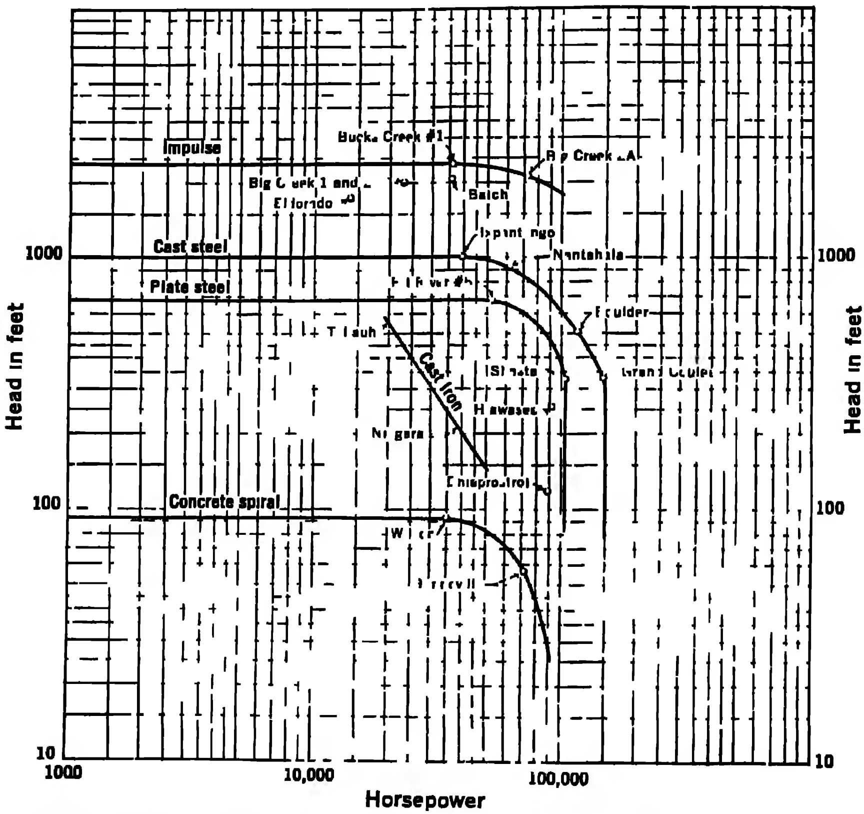


FIG 27 Usual limiting capacities and heads for various types of turbine settings

In open-flume settings for Francis or propeller wheels in which the water is not guided by partitions, the inlet velocity in the flume should not exceed 7½% of the spouting velocity of the water under rated head for full load. In any event the velocity should not be greater than 3 ft per second at full load.

With concrete scroll casings, the velocity around the circumference should not exceed about 14% of the spouting velocity at full load, but maximum velocity should not exceed 10 ft per second. Velocities of 5 or 6 ft per second are recommended where practicable.

The velocity at full load in plate-steel scroll casings may be as high as 16% of the spouting velocity, but should perhaps not be over 20 ft per second With smooth casings of cast iron or cast steel a full-load velocity up to 20% of the spouting velocity may be used, with a maximum velocity of 30 ft per second being allowable for heads of 400 ft or more

A usual design of an open-flume vertical-shaft setting for a Francis or propeller turbine is shown in Fig 25 Note that the corners of the flume have been rounded, which somewhat improves the velocity conditions within the

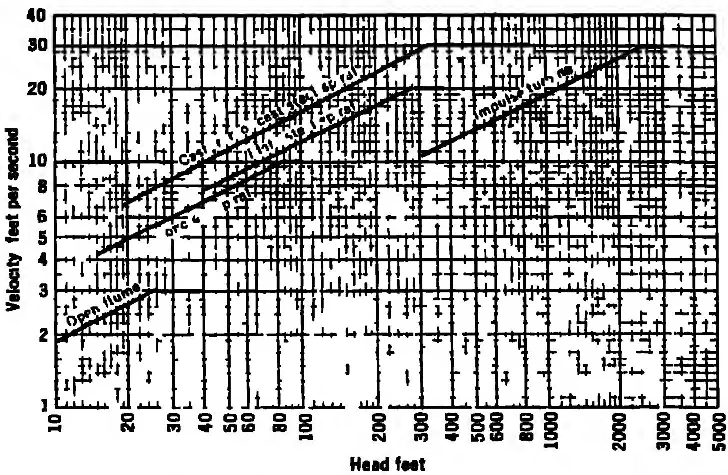


FIG 26 Curves showing allowable water velocities in various types of casings for reaction wheels and inlet pipes of impulse wheels

flume Trash racks are provided outside the powerhouse Head gates or stop-logs may be located outside or may be inside the powerhouse for handling with the powerhouse crane For the circular shape of flume wall shown, a straight wall may be substituted without introducing appreciable hydraulic losses provided the flume is of ample area The turbine is shown set close to the flume floor but it is often raised somewhat higher The plate-steel concentric type of draft tube here shown has the advantage of being low in cost as it obviates concrete formwork The generator being placed safely above the maximum headwater level dispenses with a bulkhead and stuffing box around both main and regulating shafts A hatchway is provided in the generator floor for lowering turbine parts into the flume The governor is located on the generator floor, and the regulating shaft extends into the flume where it is connected to the turbine-gate shifting ring by means of two reach rods The hydraulic torque produced by the water flowing through the guide vanes and transmitted to the cover plate is taken up by braces from two sides of the flume

It is important in the design of an open-flume setting to locate the wheel at sufficient depth below the water surface so that vortices will not be formed and air be drawn into the wheel, resulting in decreased power and efficiency and possibly in heavy vibration. The depth of the water above the distributor should be at least equal to, and preferably several times, the runner diameter.

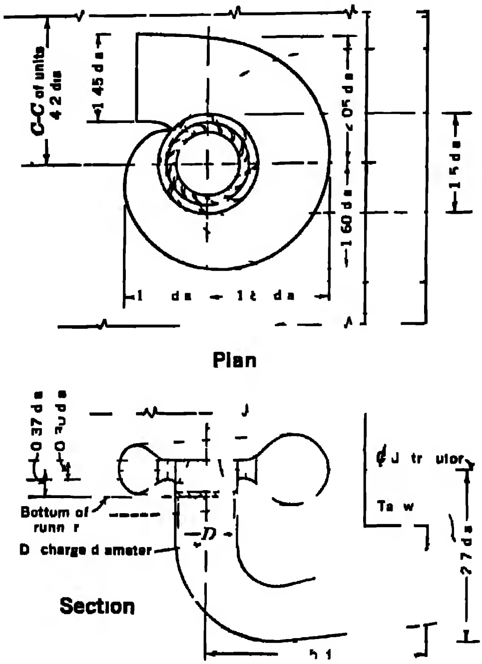


FIG. 29. Approximate setting dimensions for preliminary estimates. Francis turbines with steel scrolls.

The scroll or spiral case is designed with a cross section area reducing uniformly around the circumference from a maximum at the entrance to the scroll to very nearly zero at the tip. This reduction results in the water's flowing smoothly at a nearly constant velocity around the casing, producing uniform distribution of water to all parts of the runner and a nearly uniform pressure around the entire casing. The direction of flow around the scroll is the same as the direction of rotation of the runner.

The approximate dimensions of the scroll case in terms of rated head, runner diameter, and specific speed are shown in Figs. 29 and 30 for Francis and propeller wheels. These dimensions represent the average for a number of successful installations and are intended to be used only as a preliminary estimate to determine approximate costs. Detailed designs should generally follow the suggestions of the manufacturer, as he is usually held responsible for the successful functioning of the unit.

Little difference in dimensions of the setting was noted for Kaplan and fixed-blade runners of the same diameter, although the Kaplan runner discharges about 12% more water at full load.

The full scroll may have a circular cross-section as shown and be of steel or reinforced concrete, or, for low heads, it may have a rectangular cross-section with rounded corners, as shown in Fig. 25, which will provide sufficiently low velocities. For large units one or two piers divide the water

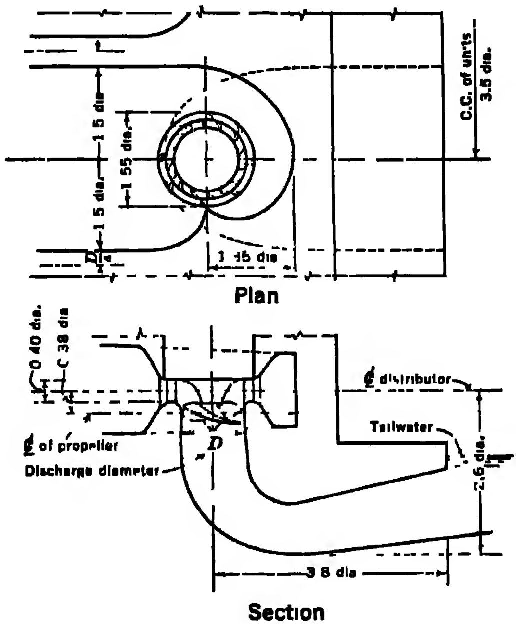


FIG. 30. Approximate setting dimensions for preliminary estimates. Propeller turbines with concrete scrolls.

passage upstream from the wheel as shown in Fig. 32; these piers serve both to guide the water and to strengthen the roof slab.

Figure 31 is a sectional elevation of a siphon-type concrete casing with a turbine of the propeller type. A plan view of this type of casing would be like that shown in Fig. 32. The water enters the concrete casing through suitable trash racks, head gates similar to those used with the open-flume setting being provided to shut off the water for inspection or when the unit is shut down for a material length of time. The water is led to all sides of the turbine, the design of the casing being such as to give, as nearly as possible, uniform velocities in all parts. The generator in Fig. 31 is shown mounted on a short supporting barrel and is reached by a short stairway, access to the turbine cover plate being had through an arch and stairs leading down into the turbine pit above the small end of the spiral. The governor is located

on the main floor, and the servomotors or regulating cylinders for controlling the position of the gates are located in the turbine pit. For large units where the governor capacity is above 25,000 or 30,000 ft-lb, the connection is made directly to the shifting ring, but units requiring smaller governors are usually equipped with a vertical regulating shaft similar to that shown in Fig. 25.

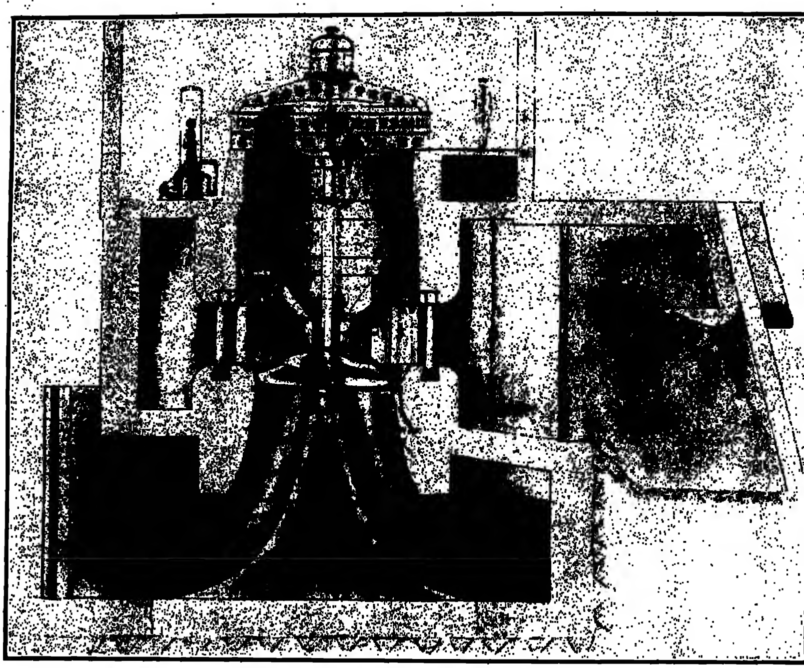


FIG. 31. Dixon plant, Illinois; propeller turbine, 800 hp, 80 rpm, 8-ft head; vertical concrete spiral-cased, siphon-setting, turbine; column-type speed ring, plate-steel pit liner, and concentric plate-steel draft tube with cone center. (I. P. Morris.)

Figure 33 shows a cross-section of one of the Safe Harbor Kaplan units. The concrete scroll case in this figure is of rectangular cross-section. This view also shows the runner set well below the normal tailwater elevation to mitigate cavitation. These wheels are rated at 42,500 hp at 109.1 rpm and 55-ft head. The diameter of the runner is 220 in. The approximate full gate discharge is 8000 cu ft per second. Note that the powerhouse superstructure is relatively small and that the draft tube extends beyond the powerhouse wall.

The Pickwick Landing units are rated 55,000 hp at 81.8 rpm and 48-ft head, each discharging 13,000 cu ft per second through a propeller diameter of 292 in., so far the largest in the United States.

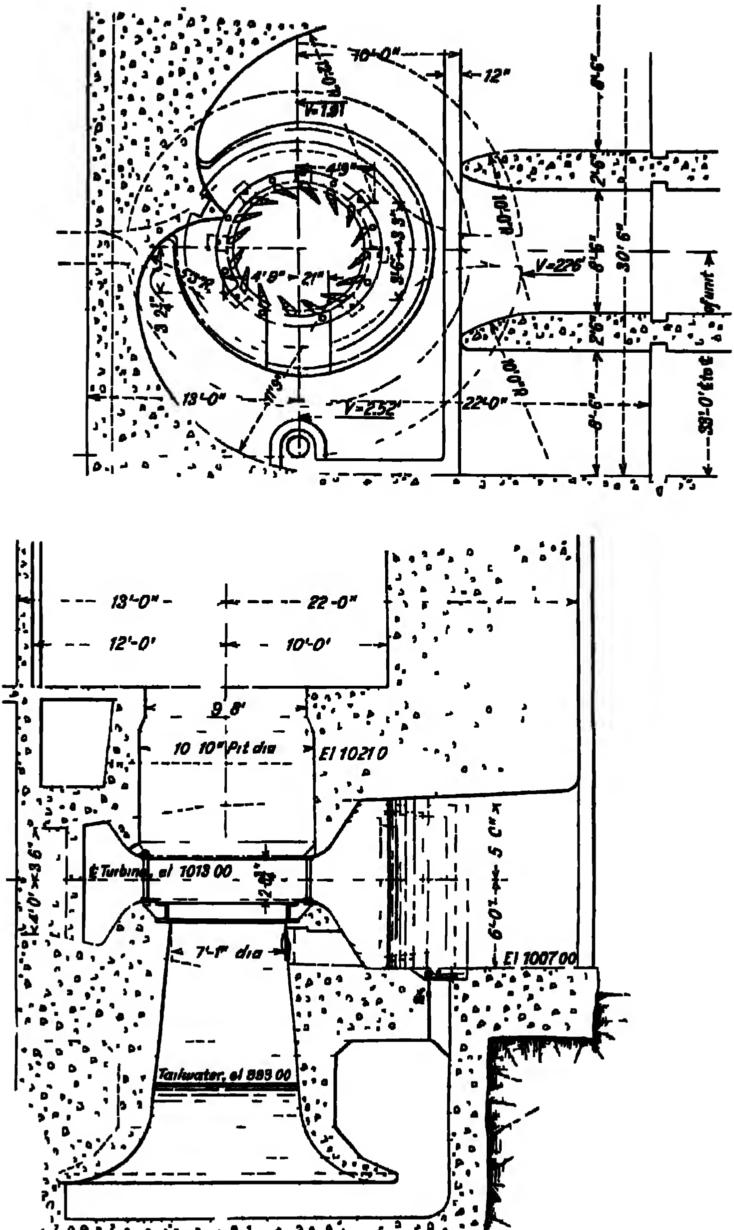


FIG. 32 Plan view and section of concrete spiral casing having nose at the rear, showing stay-bolt-type speed ring, drain valve, draft-tube liner with inspection manhole, and concentric concrete draft tube

A plate-steel casing (see Fig. 34) is usually made of segments of steel plate fastened to each other and to the speed ring (Chapter 39, Section 2) by riveting or welding. For the larger sizes, the plate segments are made of a series of developable cones to provide the desired taper. In the smaller sizes, the plates are usually pressed or hammered to form a rounded, continuous spiral. This method is more expensive than the rolling of the developable cones but is desirable for small sizes.

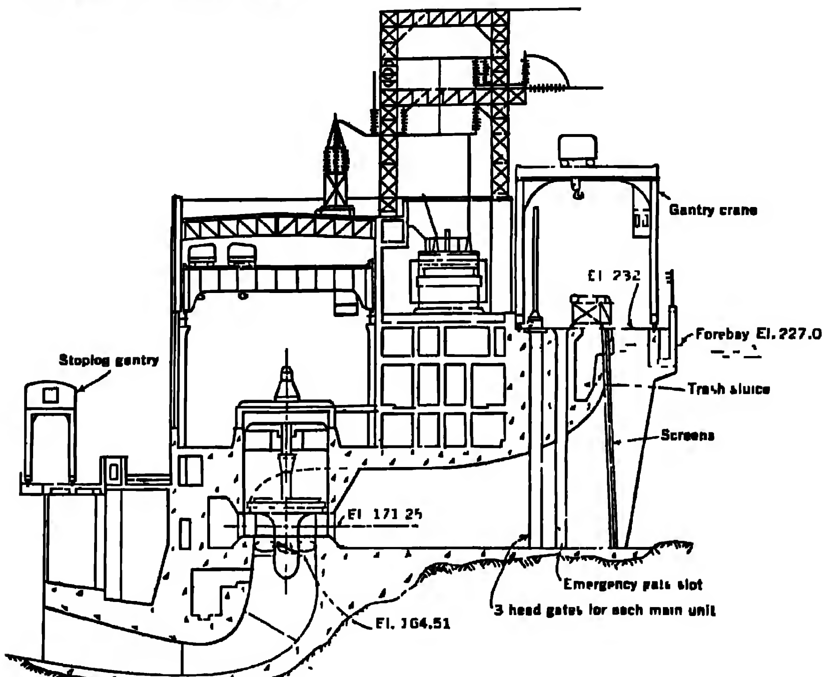


FIG. 33. Section through intake, Safe Harbor Hydroelectric development, Pennsylvania. 42,500-hp Kaplan turbines 55-ft head, 109.1 rpm. (See Table 1.) (I. P. Morris and S. Morgan Smith.)

For the cone-type casing the plates are usually punched and rolled and then fitted together in the shops of the manufacturer, together with the speed ring, the rivet holes all being reamed and the plates carefully match-marked so that they may be readily assembled in the field, when they are riveted and calked. The joints are either single- or double-riveted, depending on the pressure, and are lapped, although butt joints with butt straps outside are used by some manufacturers.

Figure 34 is a section through the 70,000-hp unit No. 21 in the plant of the Niagara Falls Power Company. This figure shows clearly the plate-steel spiral casing, the cast-steel speed ring which holds the edges of the casing

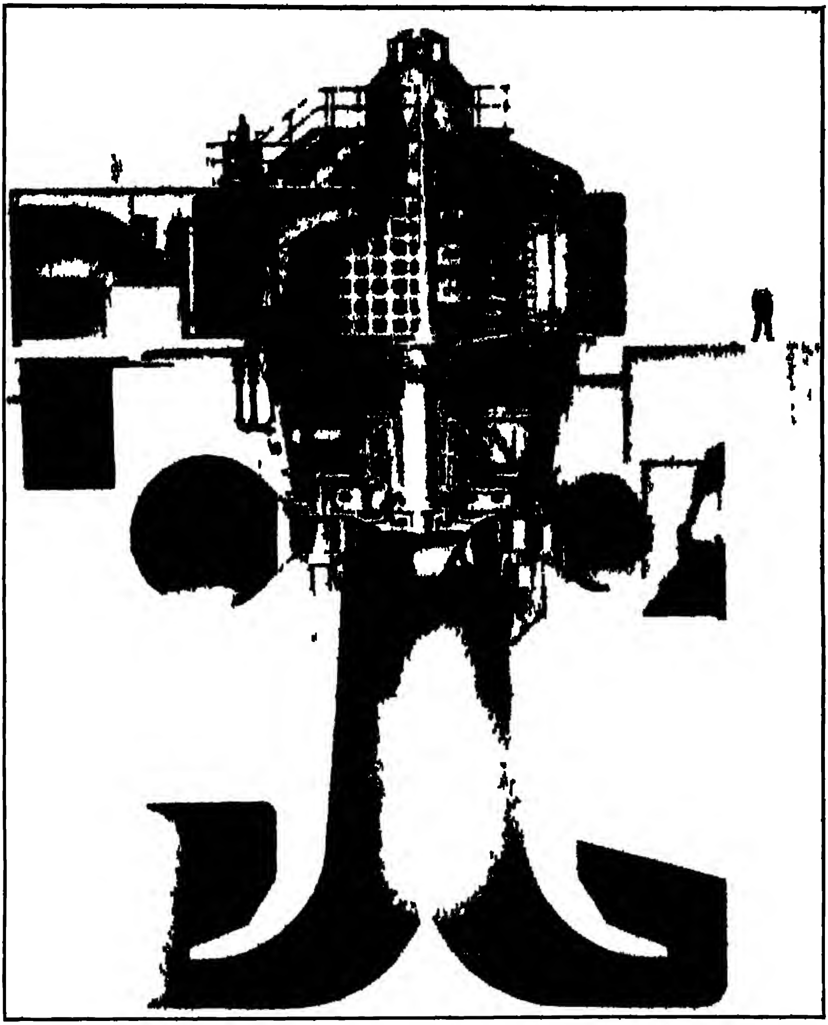


FIG. 34 Niagara Falls Power Co., New York, 70 000-hp Francis turbine unit No 21 214-ft head, 107 rpm Concentric draft tube of concrete with low-cone center, cast-steel runner bolted to flange on shaft plate-steel scroll case, adjustable lignum vitae bearing steel sleeve on shaft cast-steel speed ring cast-iron pit ring and supporting barrel carrying weight of generator, direct-connected governor flyballs, regulating cylinders bolted to pit ring and connected directly to opposite sides of shifting ring Kingsbury thrust bearing above generator, turbine bearing serves as lower generator bearing, making two-bearing unit (Allis-Chalmers)

together, and the guide vanes and operating mechanism which control the flow of water to the runner. The draft tube shown here is of the concentric concrete hydraucone type with a plate-steel liner in the upper part. In this case the weight of the generator is carried on a cast-iron supporting barrel, which transmits the load through the speed ring of the turbine to the foundation. This unit has only two main guide bearings, the one located on the cover plate of the turbine, the other just below the thrust bearing on the bridge above the generator.

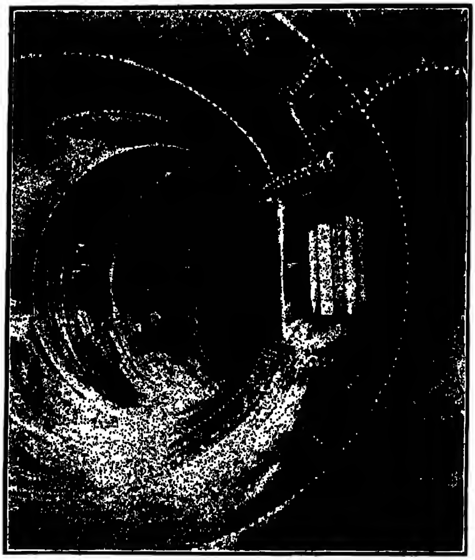


FIG. 35. Photo looking into 10½-ft inlet diameter plate-steel spiral casing of 2600 hp, 40-ft head, 150 rpm. showing speed ring with guide vanes in closed position. Single-riveted casing joints with double riveting at junction with cast-steel speed ring.

Figure 35 is a photograph taken inside a plate-steel spiral casing and shows the method of connecting the separate plates which make up the casing, as well as the joint between the plate-steel casing and the speed ring. The first opening through the speed ring where the water enters the guide vanes is clearly shown. Round-headed rivets are shown here holding the plates together, but more recent practice, especially on the higher head plate-steel casings where the water velocities are greater, has been to use the flat-headed countersunk rivets which offer very little obstruction to the flow.

Figure 36 shows a welded circular-section plate-steel spiral casing for the four 103,000-hp, 330-ft head, 138.5-rpm units for Shasta Dam, California, of the U. S. Department of the Interior. For a test pressure in the field after erection and with an inlet diameter of 152 in., a cast-steel casing involves

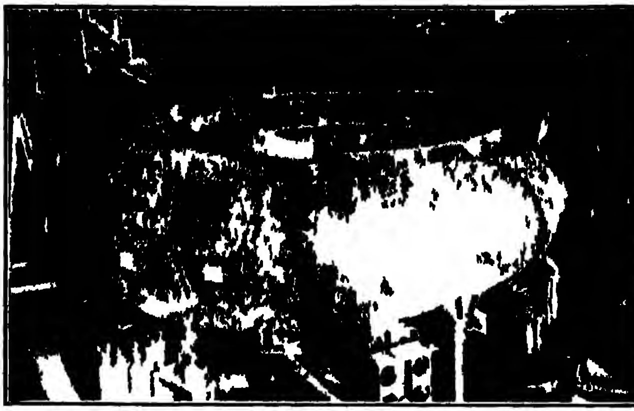


FIG 36 Welded steel-plate spiral casing Shasta power plant, California, 103 000 hp, 330-ft head, 138 5 rpm

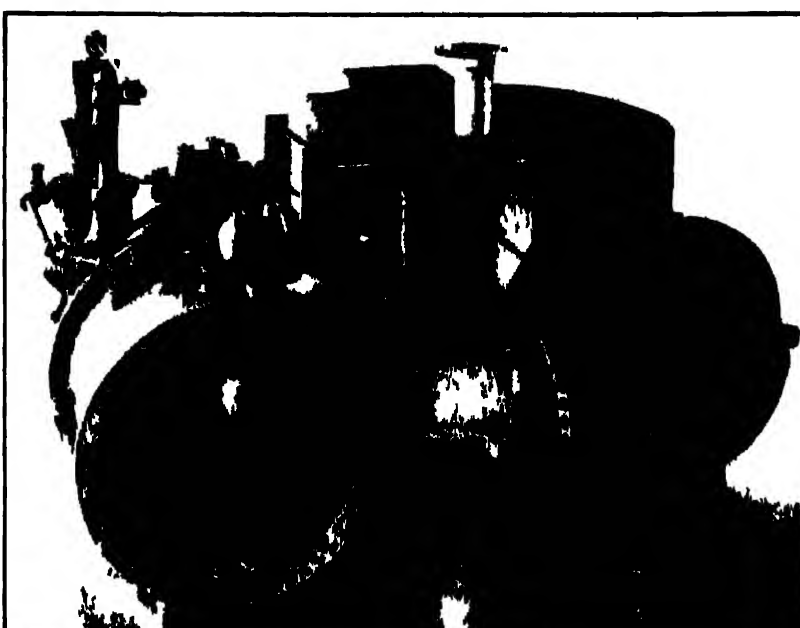


FIG 37 Cast-iron spiral-cased turbine casing made in halves with separate speed ring Cast-iron pit liner and servomotors bolted to pads on casing, pressure regulator governor operated

thicknesses of metal which also introduce quite a problem as to size of flanges and bolts. A careful analysis of costs pointed to a material saving in favor of welded plate steel. Plate-steel casings are usually shipped knocked down and are riveted or welded together and to the central cast-steel speed ring in the field.

The welded design permits of welding the plates to the speed ring at manufacturers' shops in as few sections as transportation permits. The Shasta casings are made in seven sections with transverse joints, the plate being held together by butt straps riveted in the field and the speed-ring portion by bolted flange joints.

Since plate thicknesses exceeding 2 in. were involved, positive assurance had to be obtained of the soundness of the material and especially of the various welds. Physical and metallurgical properties of steel plate and union melts were obtained from numerous specimens before and after stress relief after welding. Micrographs showed no apparent change in the grain structure of the weld before and after stress relief. Impact tests of the specimen were also made. All the results prove beyond question that a welded casing is in every respect equal and in many respects superior in quality to a cast-steel casing. Figure 36 shows the six butt-strap joints of the casing sections proper.

The continued improvements made by steel mills in the quality of plate-steel material permits of allowing higher design stresses than have heretofore been considered good practice. Thus heads exceeding 400 ft are now within practical range. Heavy plates require correspondingly heavy rivets, thereby setting limits. New data made available by the rapid advancement in the art of welding have removed any doubt as to the feasibility of welding casing sections together, rather than riveting them.

Since steel-plate scroll cases for vertical units are usually imbedded in concrete and are, consequently, difficult to place, allowance for wear and corrosion should be made in selecting plate thickness, and conservative unit stresses should be used.

With cast-iron scroll cases the inner circumference of the casing is held together by a series of steel stay bolts which are usually made a permanent part of the casing. Where silt or sand is carried in the water, these bolts are subjected to considerable wear and should be designed to be readily renewable. Cast scroll cases are shown in Figs. 37 to 41, inclusive.

It is not advisable to use a cast-iron casing with the speed ring (Chapter 39, Section 2) made an integral part, because of the lack of ductility of cast iron. There are several instances on record of serious damage done when abnormal water hammer caused breakage of the tension members of the cast-iron casing, whereas many cast-steel casings have successfully withstood heavy water hammer. Figure 37 shows a cast-iron casing where the speed ring was cast separately and the casing bolted to the speed-ring flanges. This casing was made in halves.

Figure 39 shows a double overhung unit where two cast-steel spiral-cased turbines are used to drive one generator, the weight of the two runners and

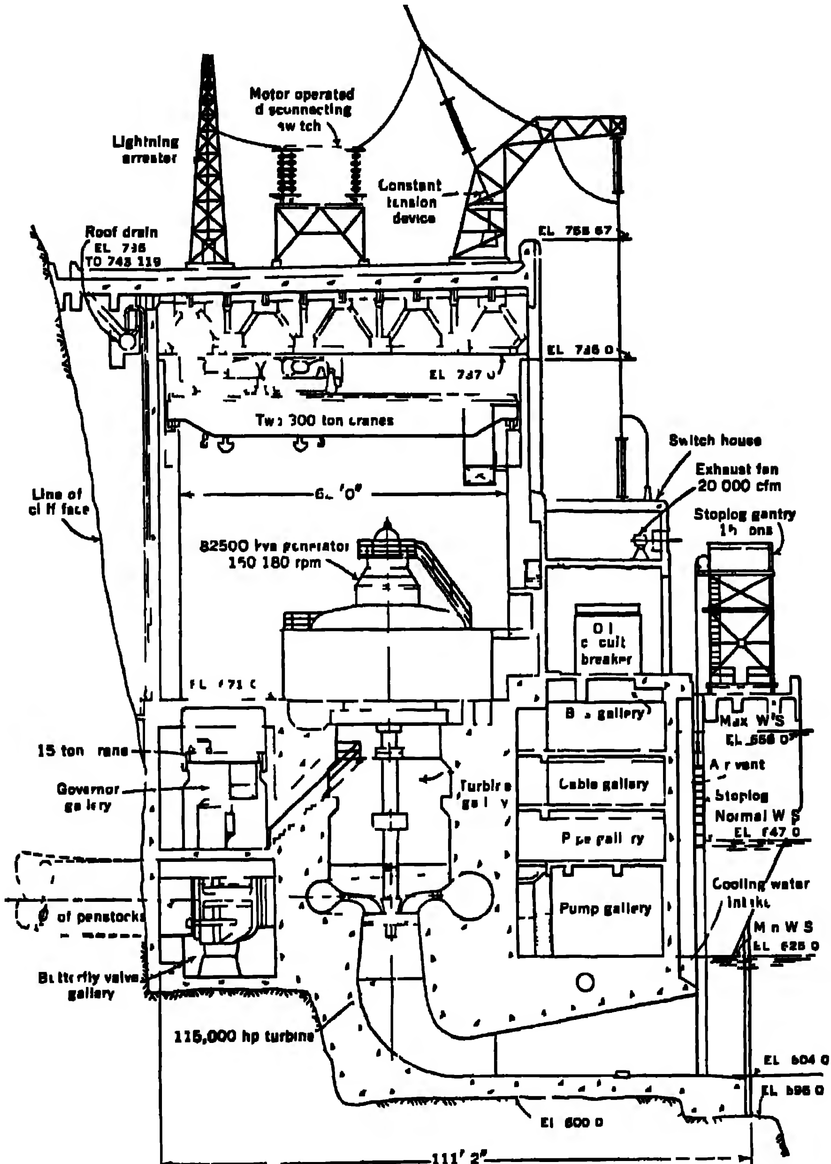


FIG. 38 Hoover Dam Nevada-Arizona, 115 000-hp Francis turbine, 510-ft head, 180 rpm (See Table 1) (I P Morris and Allis-Chalmers)

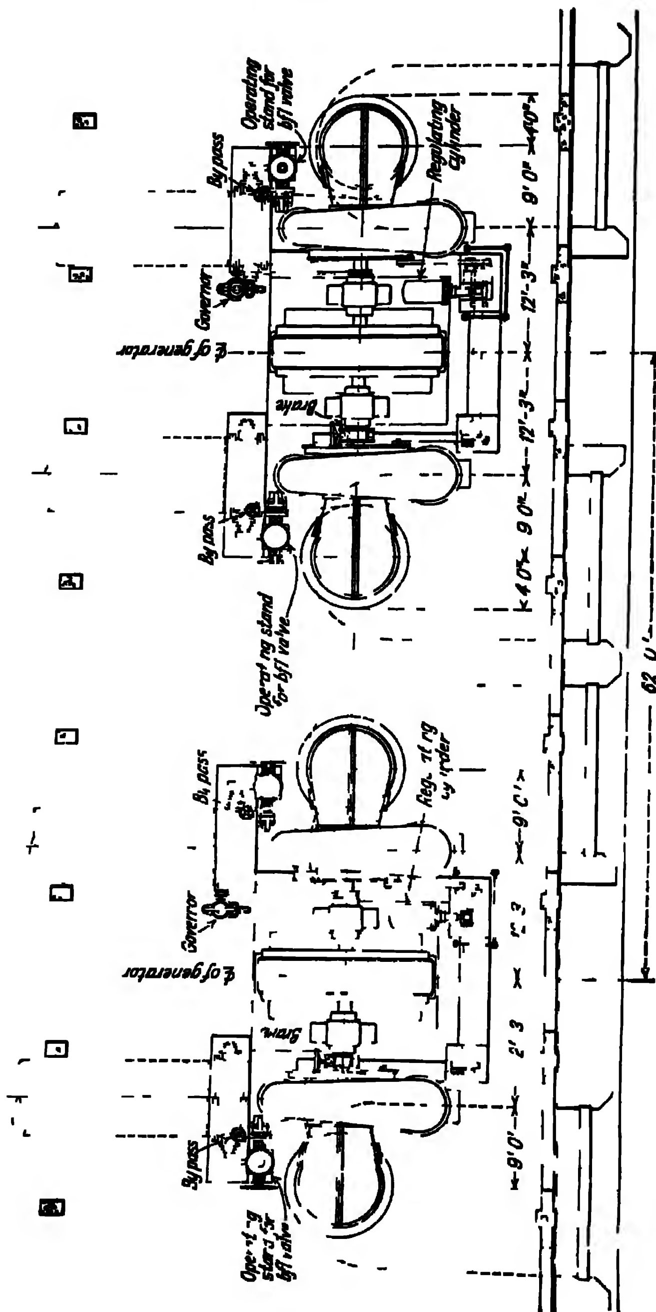


Fig. 39 Baker River plant Washington Plan view of two 20 000-hp 200-ft head double-overhung cast-steel spiral-cased turbines showing butterfly valves governor, and regulating shaft Generator located between bearings with one runner overhung from each end of main shaft (Allis-Chalmers.)

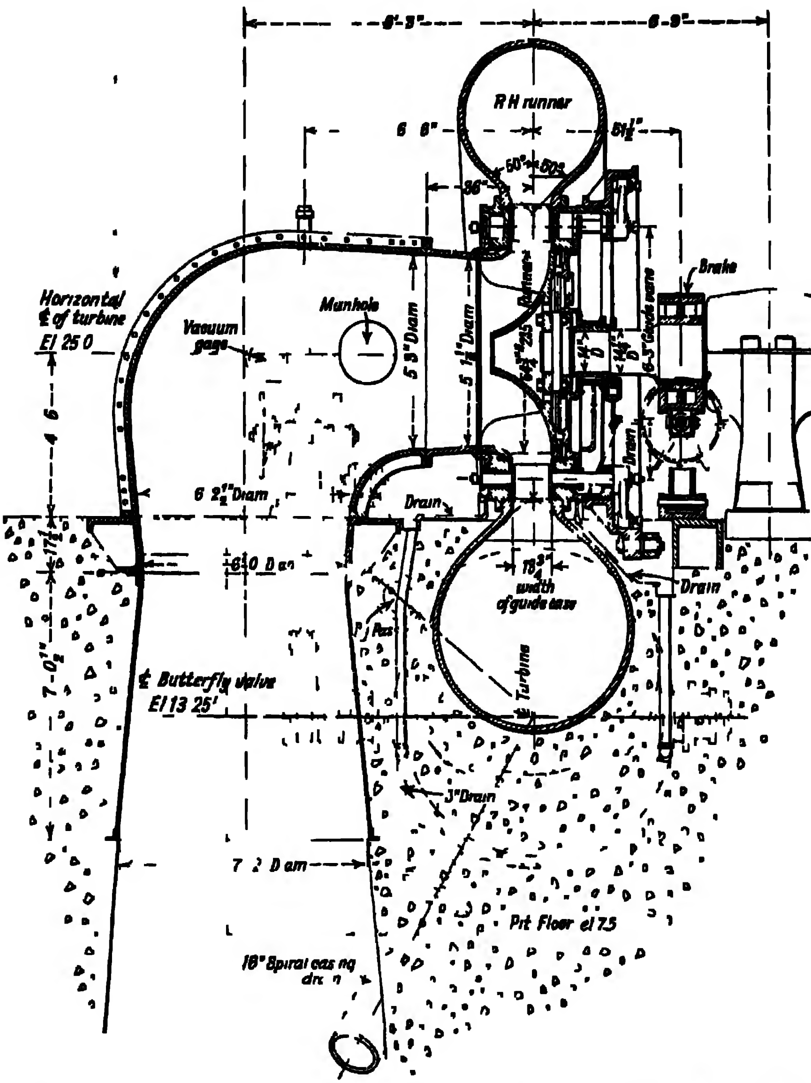


FIG. 40. Baker River development, Washington. Section through 20,000-hp 200-ft head double-overhung cast-steel cased turbine showing quarter-turn draft tubes, cast-steel runners bolted to shaft flanges so that runners may be removed without disturbing shaft, shifting ring located outside of guide-vane stems and friction brake to stop unit. (Allis-Chalmers.)

the generator rotor being carried on the generator bearings. These units may also be equipped with pressure regulators, and each unit may have its separate governor system so that either turbine may be run independently, provided thrust bearings are constructed for this purpose This permits of much more efficient operation below half load, as is the case with the double overhung impulse wheel Figure 40 is a section through one of these horizontal turbines showing the arrangement of the various parts

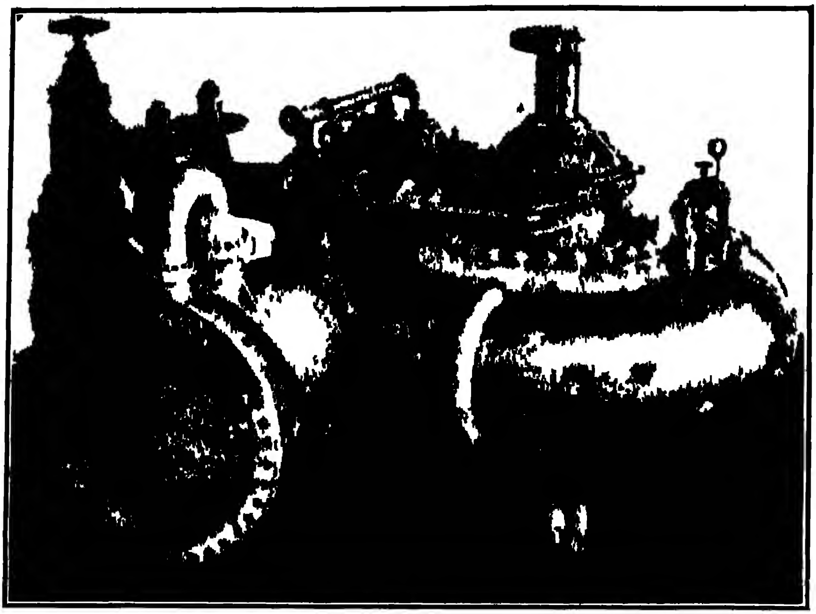


FIG 41 Oak Grove plant, Oregon Assembled view of high-head cast-steel, spiral-cased turbine 850-ft head 35 000 hp 21 specific speed Butterfly valve governor-operated pressure regulator servomotors bolted to pads on casing main steady bearing of oil-lubricated babbitted-type with auxiliary motor-driven pump mounted above bearing (Pelton)

Cast spiral-cased turbines usually have the water brought in through long penstocks under high pressure, which makes it very desirable to provide some means for shutting off the water at the inlet of the casing

Cast-steel casings should not usually be made less than ¾ in thick, owing to the difficulty of casting thinner sections of cast steel Figure 41 shows a cast-steel casing for a 35,000-hp unit operating at 850-ft head Because of the size and head, the design of the casing required special care It is made in halves, and the speed-ring ribs are cast integral with the casing, as is general practice with cast-steel spiral casings

20. Draft Tubes. In as much as the turbine manufacturers are usually required to guarantee performances and efficiency, they will naturally insist

that draft-tube design shall follow their suggestions. In fact it is usual practice for the turbine manufacturer to furnish the hydraulic design for draft tubes and scroll cases not constructed at their own shops.

The function of the draft tube is to conserve for conversion into power by the turbine as much of the remaining head from turbine to tailrace as practicable. Thus the draft tube takes the water discharged by the turbine at high velocity and then reduces the velocity as gradually as practicable by

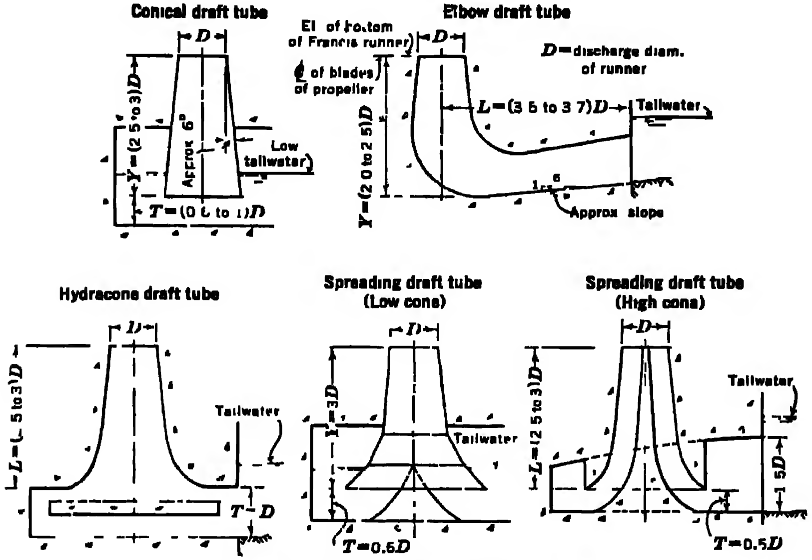


FIG 42 Typical types of draft tubes

enlarging the cross-section to a low velocity at draft-tube exit below tailwater elevation Velocities just below the runner may be 25 or 35 ft per second and at draft-tube exit into the tailrace 5 or 6 ft per second. Draft head (the H_s of Section 7) varies in general from −8 ft to +15 ft. Between a poorly designed draft tube and one of good design there may be as much as 2% or more difference in over-all efficiency. On the other hand complicated draft tubes may not be able to justify the resulting moderate gain in efficiency because of the added construction cost.

The exit to the draft tube is frequently well below the river bottom. Very gradual upslopes involving a large amount of excavation have been used to connect the floor of the draft tube to the bottom of the tailrace. A high cost for such work is not justified, as any slope flatter than a 1 vertical on 6 horizontal, which is about the upcast for some draft tubes, shows no appreciable gain. This was verified in a series of tests, made by Professor Charles M. Allen of Worcester Polytechnic Institute, which showed that an upslope even

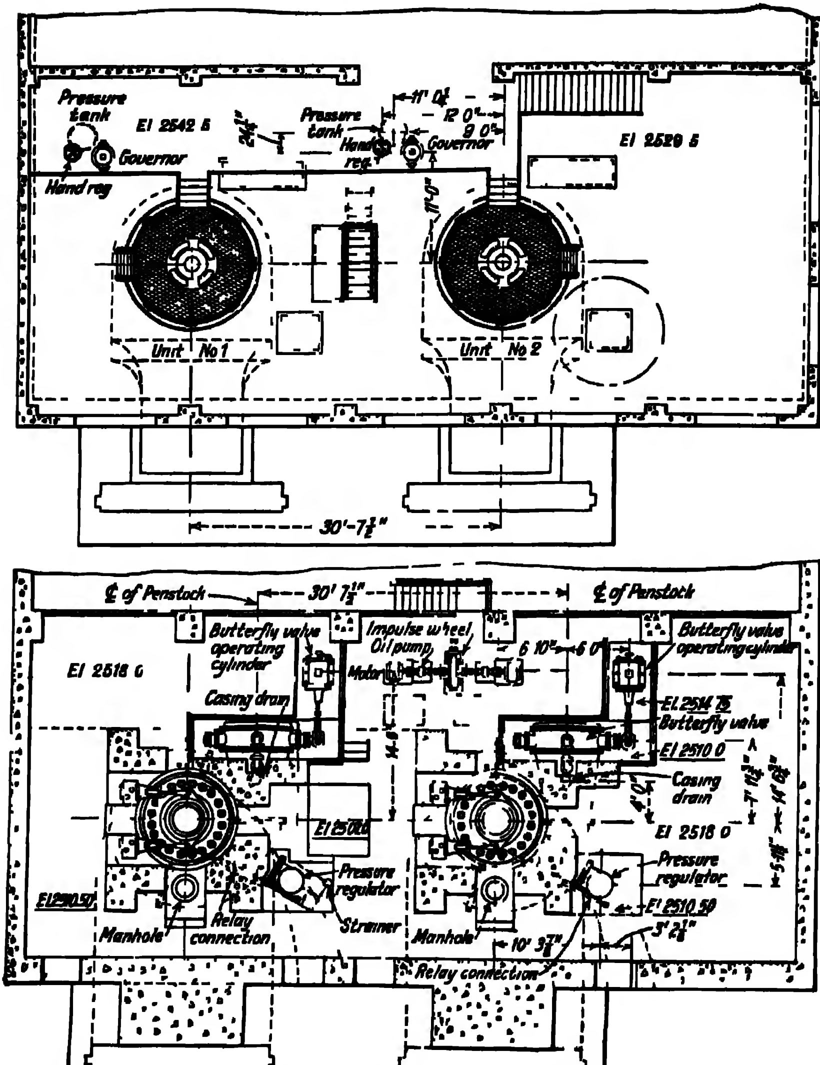


FIG 43 Vertical cast-steel spiral-cased turbine for high head, 18 000-hp, 420-ft head, 375 rpm Keihin Denryoku Electric Power Co Japan, showing three-floor arrangement with switchboard floor at exciter level main floor at generator base, and turbine floor at cover-plate level with an inspection floor below turbine Penstock enters horizontally with hydraulically operated butterfly valve Cast-steel casing made in three sections pressure regulator with plate-steel discharge pipe, concrete-elbow draft tube with removable cast-iron section below runner Governor stands and controls located on switchboard floor motor and impulse wheel-driven oil pumps located on turbine floor (Allis-Chalmers)

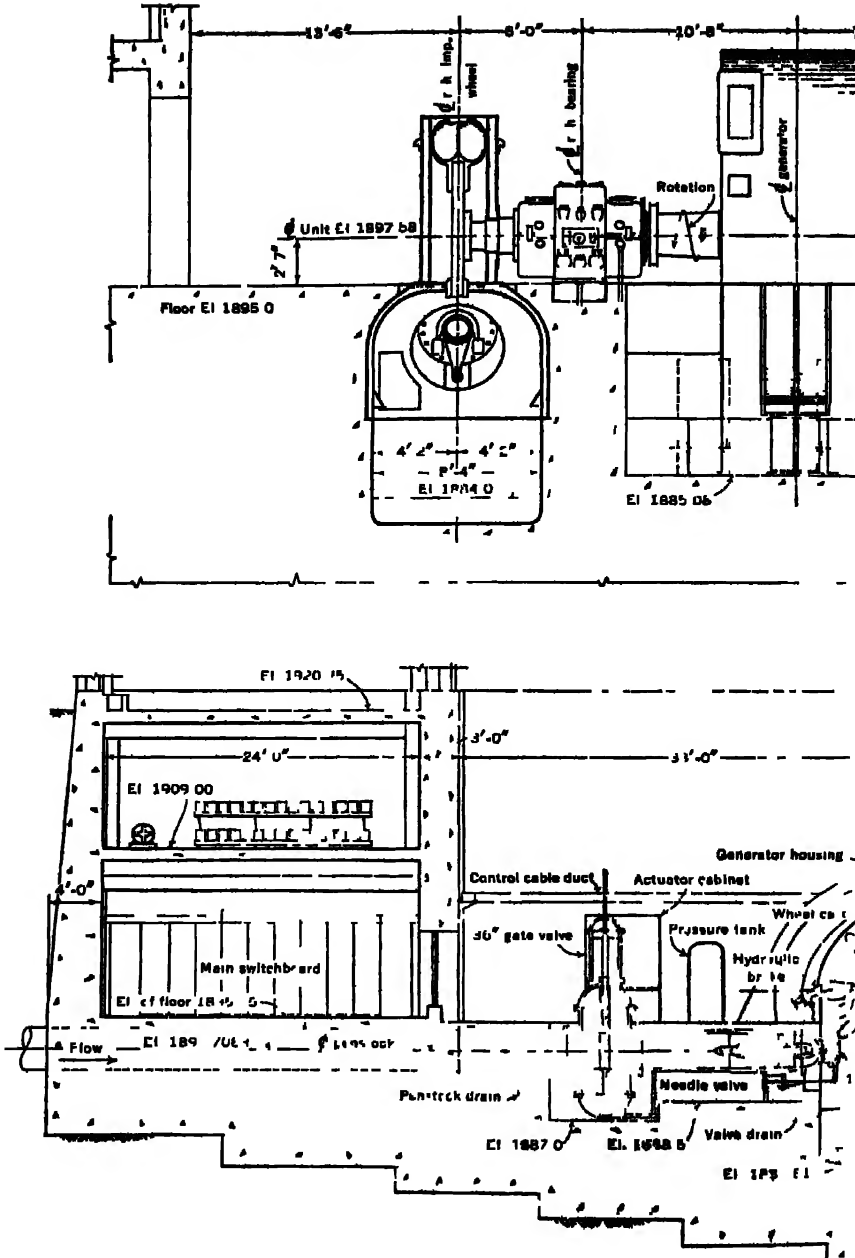
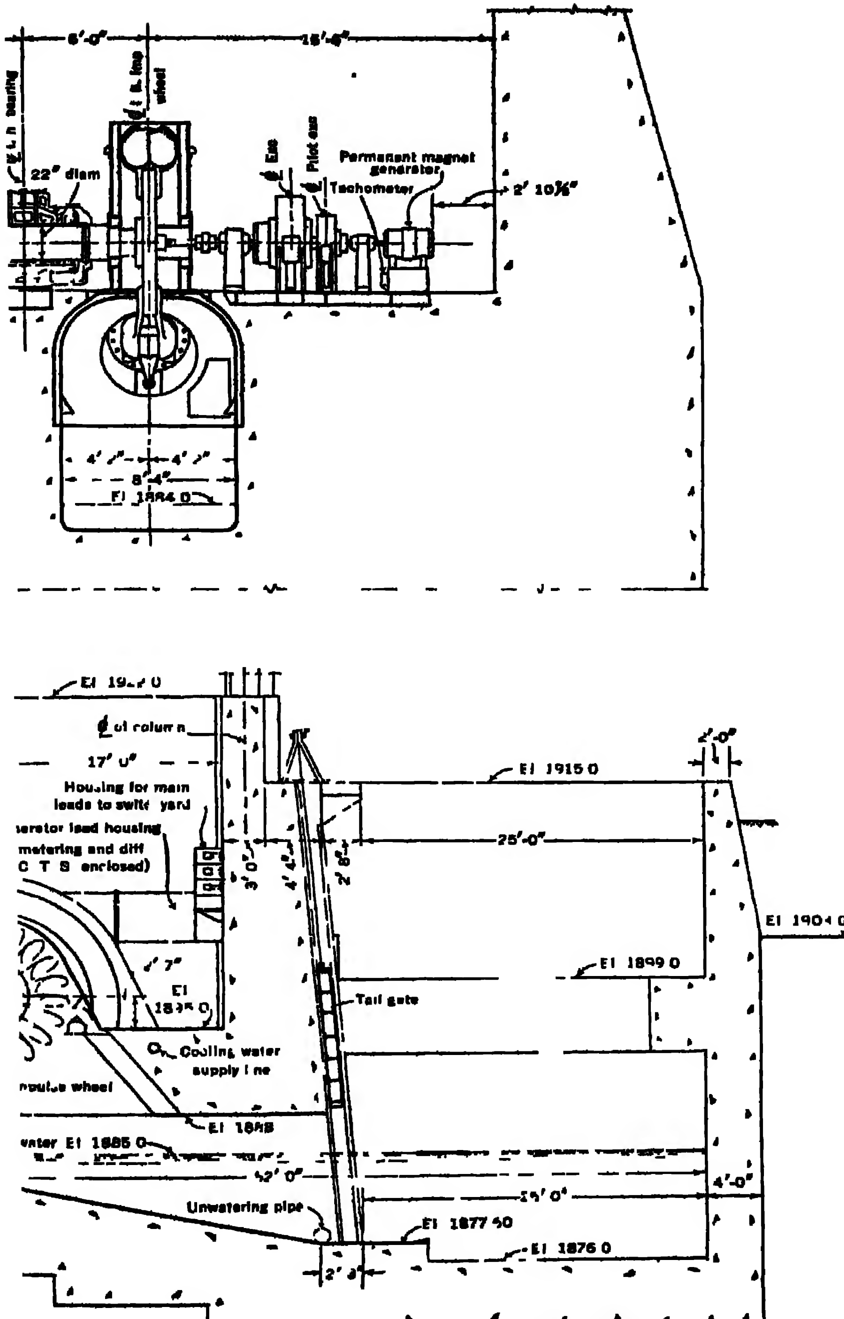


FIG. 44 Glenville, North Carolina; 30 000-hp, 1150-ft head, doubl



ting single-nozzle, 257-rpm impulse wheel. (Allis-Chalmers.)

as steep as 1 on 4 beyond the draft-tube exit resulted in practically no greater loss of head than a very gradual upslope to the bottom of the tailrace.

The types of draft tube and their usual dimensions as used in modern installations are shown in Fig. 42. The two most common types are the straight conical draft tube and the elbow type of tube. The conical tube consists of a straight cone having a flare of 4 to 7 degrees from the center line. The greater angle of flare is less efficient, unless provided with a flare at the discharge end, either of the Moody spreading type or the White hydraucone type.

The elbow type offers advantages in cost of excavation. Its length is usually between 2.5 and 3.7 times the throat, or runner discharge, diameter. The elbow type has become the conventional draft tube for large units, being constructed usually of concrete with a plate-steel liner in the upper portion where pressures are usually negative. A fin or dividing wall is sometimes put into the elbow section to improve the flow and efficiency and for structural reasons in large units.

Draft-tube Liners. Except on relatively small units under low heads, it is desirable that at least the upper portion of the draft tube be lined with either a cast- or plate-steel liner. The water discharging from the runner has a rapid whirling motion, and where unlined draft tubes have been used it has frequently happened that the concrete has been eaten away by pitting to a depth of 12 to 18 in. just below the runner so that it seriously decreased the strength and endangered the fastening of the discharge ring and other turbine parts.

Draft-tube liners can be made readily of plate steel riveted or welded together. They should be provided with flanges at top and bottom and anchor straps so that they will hold firmly to the concrete. Either a cast- or structural-steel flange or other permanent field joint should be used at the top, and this should be bolted to the turbine proper.

On high-head units where the velocity in the draft tube may be high the upper section of draft tube is usually made of steel plate or cast steel with manholes provided on opposite sides to permit inspection An interesting design of draft-tube top is of the telescopic type, shown in Fig. 43, where this upper cast-iron section is so arranged that it may be lowered directly into the lower part of the draft tube, thus leaving a clear space for inspection purposes when in the lowered position. This telescopic section is bolted to the bottom of the discharge ring, and a packed joint is used at the lower joint to prevent leakage of air into the draft tube. This and similar constructions have been used extensively and have resulted in a material saving in time, especially where frequent changes of the runner were required, such as might be due to silty or bad water conditions.

21. Impulse-wheel Settings. The most usual arrangement in the United States is the horizontal-shaft double-runner overhung type, as in Fig. 45, having an impulse wheel placed in an overhung position on each side of the generator, each wheel self-contained so that the water can be shut off entirely

on one side if desired. This type, using an independent governing equipment for each side, has become standard design on account of its flexibility.

With impulse wheels, the full-load velocity in the nozzle pipe ahead of the nozzle tip is usually made less than 10% of the spouting velocity under the full rated head for the wheel. A better value is 7½% with not over 30 ft per second maximum. Higher velocities cause distortion of the jet where there are bends in the nozzle pipe to provide for the operation of the power needle valve.

FIG. 45. Big Creek 2A, California; 56,000-hp, double-overhung, single-nozzle impulse turbine, 2200-ft head, 250 rpm. (Allis-Chalmers.)

In some designs two or more jets are disposed around the periphery of a vertical-shaft impulse wheel in order to obtain increased generator speed. This practice is particularly common on European wheels designed for export, but it is not general in the United States. Multiple-jet units are usually less efficient than single-jet units because of interference of water on the buckets.

With impulse-wheel units such as those illustrated in Figs. 44, 45, and 46, the flow of water is controlled by a needle valve in accordance with the load. With long pipelines dangerous pressure surges are likely to occur if the momentum of the moving water column is reduced fast enough to hold the speed of the unit within desirable limits. In such cases an auxiliary outlet is provided which opens as fast as the flow is reduced by the power needle so that the flow in the pipe is not suddenly reduced. By means of a dashpot the auxiliary outlet is closed gradually, thereby reducing the flow in the pipeline. When load is thrown on again, it can be picked up only at the rate at which the water can be accelerated in the pipe without resulting in an undesirable pressure drop, thus avoiding an augmentation of the resultant speed drop of the unit. Flywheels are sometimes used to better this condition. Auxiliary

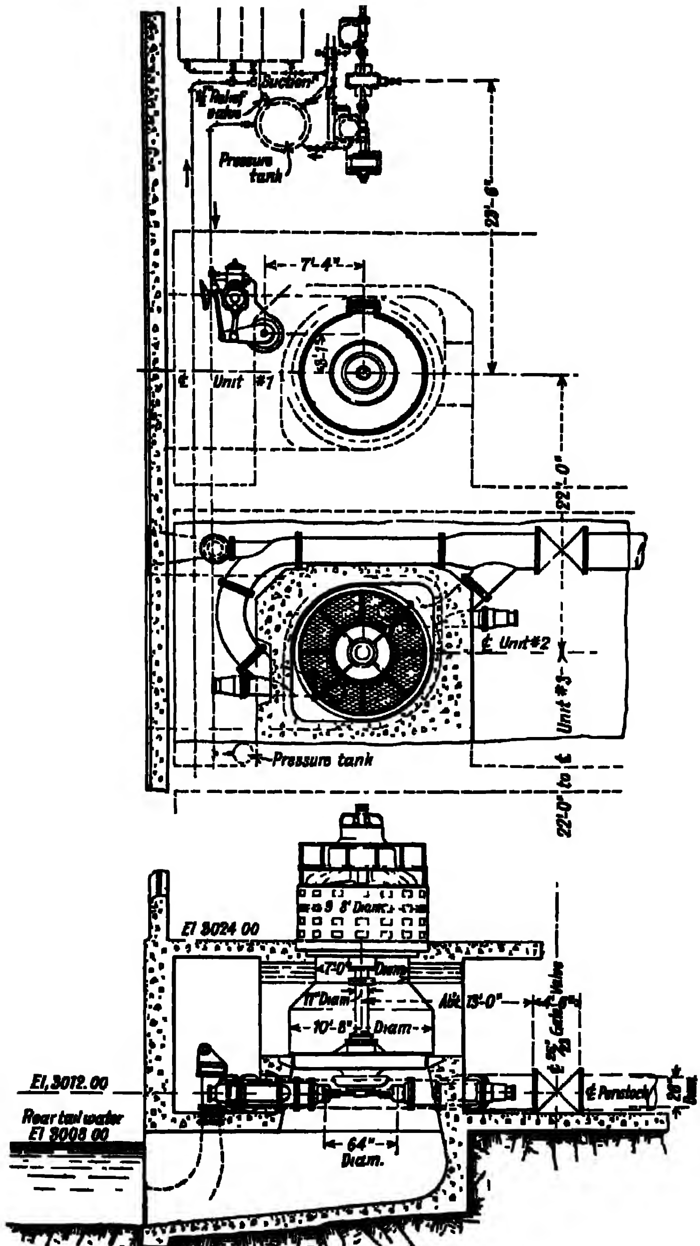


FIG. 46. Typical vertical-shaft two-jet impulse wheel, both needles operated from one governor. Governor-controlled pressure regulator, two-floor station.

outlets are also used in connection with Francis turbines supplied through long penstocks.

Where pipelines are very long, or where water must be discharged at all times to the tailrace, as with irrigation plants, the auxiliary outlet is then

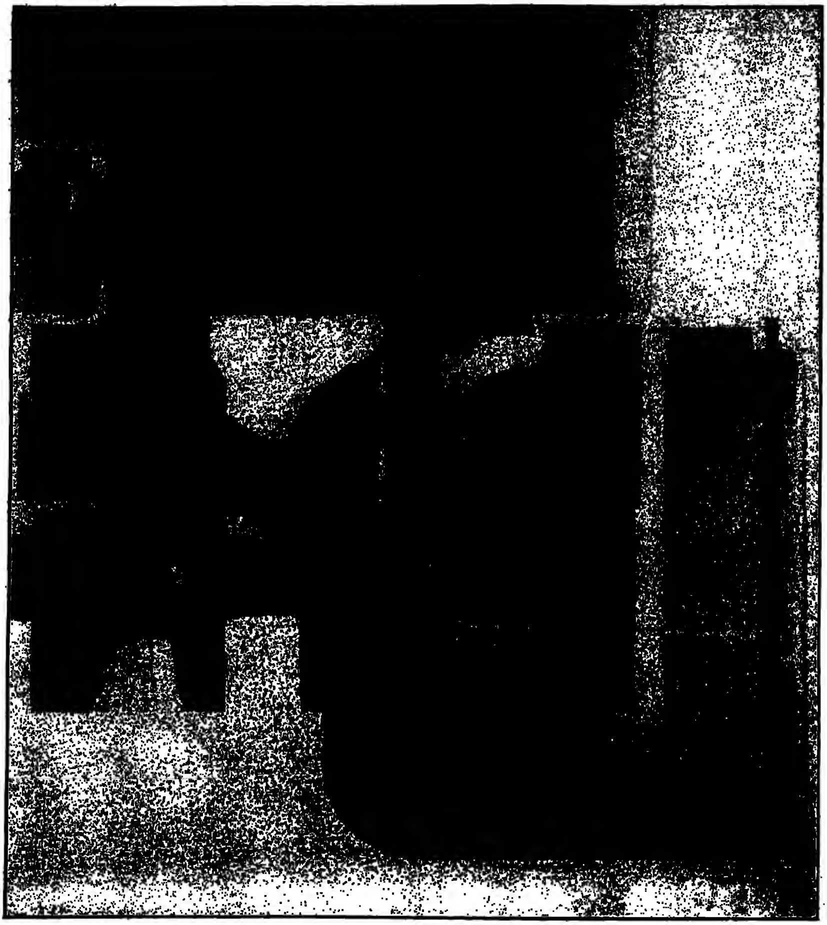


FIG. 47. Pit River No. 5 plant, California; 50,000-hp, 587-ft head, 300 rpm, vertical reaction turbine. (See Table 1.) (Pelton.)

directly connected to and operated in synchronism with the movement of the power needle.

Instead of using an auxiliary outlet many impulse units with long penstocks use deflecting nozzles where the power needle is set for a desired constant discharge and where the jet is merely deflected from the bucket wheel.

Instead of deflecting the jet by means of the hinged deflecting nozzle, in the standard design the nozzle pipe and its needle control mechanism remain stationary and a sleeve-type deflector is provided between the bucket wheel and the nozzle orifice. This sleeve is actuated quickly by the governor, deflecting the jet partly or wholly on load rejections.

Where water-saving action is required the needle is interconnected either mechanically by means of a dashpot or hydraulically in such a manner that it follows the movement of the quick-acting jet deflector, at a rate, however, which prevents undesirable pressure variations in the pipeline.

Figure 44 shows a single overhung horizontal impulse turbine having one jet operating on the buckets and disk. The buckets and disk are mounted on the extended end of the generator shaft, the two generator bearings carrying the weight of the rotor and the impulse wheel. On this unit the regulating cylinder of the governor is connected directly to the needle, a governor-operated pressure regulator being provided so that when the needle is closed rapidly the pressure regulator opens simultaneously, preventing a serious rise in pressure. The pressure regulator then is caused to close slowly so that no more water will be wasted. Some reliable type of valve is usually required in an impulse-turbine plant as the penstocks are usually long and the pressure high. The majority of plants use a special type of gate valve, although there are some installations where the needle type of valve is used. (See also Chapter 39, Sections 17 and 18.)

Figure 46 shows an arrangement of a vertical-shaft impulse wheel where two jets are used on the one disk, a pressure regulator being provided to prevent serious pressure rises. The generator is arranged similarly to that for a cast-casing reaction wheel, the water being brought into the wheel through a penstock located below the floor. The revolving parts of the impulse wheel are covered in a waterproof chamber.

22. Efficiency Tests. Efficiency tests of turbines in hydraulic power plants are desirable and furnish valuable information regarding the performance of the power-generating equipment With new installations, such tests determine whether the manufacturer's guarantees of efficiency and power have been met. If the efficiency is lower than the guarantee, there may be a serious economic loss over the period of the life of the turbine [20].

With old units, efficiency tests help to indicate whether the existing equipment should be replaced with more modern and efficient apparatus to utilize the available flow more economically. Often, as a result of testing, inefficient operating practices are discovered and corrected. If the efficiency of all or most of the units in a system is known the distribution of the load among the various generating units of the system in the most economical manner is greatly facilitated. Accordingly, periodic check-ups are often desirable. This is a very simple matter if some method of permanently measuring the discharge of the turbines is available. Otherwise it may be desirable to make complete efficiency tests on a unit at intervals of several years.

The operation of adjustable propeller blades in conjunction with the governor is usually checked at the time of the acceptance field tests to insure that the most efficient blade settings are used.

A complete efficiency test of a hydraulic turbine requires accurate measurements of the power output, head, and discharge at a sufficient number of gate openings so that curves of these values may be plotted. Such tests, to be reliable, should be made or supervised by men experienced in the various departments, as the conclusions drawn from test results are sufficiently important to warrant all possible care in the performance of the test. In tests of new units where there is an efficiency guarantee with or without penalties for nonperformance, the test procedure and the personnel in charge of the test should receive the sanction of the manufacturer. Also, a representative of the manufacturer should be present during the test.

In modern hydroelectric plants the power output of the turbine is usually measured electrically by measuring the output of the generator, using calibrated test instruments, transformers, and leads. Properly determined allowances are made for the various generator losses (see Section 5, Chapter 41).

The effective head on the turbine, on which efficiency computations are based, is the vertical distance from headwater to tailwater minus all losses down to the entrance of the scroll case and minus the velocity head in the tailrace at a point just below the draft-tube exit. (See Fig. 1, Chapter 9, "Head, Power, and Efficiency.") Thus, for purposes of determining the efficiency of the turbine, only the losses in the turbine-casing, losses in the turbine itself, and the losses in the draft tube are considered

The American Society of Mechanical Engineers Test Code (1938) provides (Paragraph 84) that

> The effective head on the turbine shall be taken as the difference between the elevation corresponding to the pressure head in the penstock at the entrance to the turbine casing and the elevation of tailwater; the above difference being corrected by adding the velocity head in the penstock at the section of measurement and subtracting the residual velocity head at the section of measurement in the tailrace.

The above applies to reaction turbines (Francis and propeller). In the case of impulse wheels, the effective head is generally taken as the pressure head at the entrance to the nozzle, plus the velocity head at this point.

The turbine discharge may be measured by one or more of several established methods as follows:

1. The Allen salt-velocity method, which utilizes the time required for a dose of salt solution to pass between two stations, the salt solution being detected by the variation in an electric current passing through the water.
2. The Gibson method, which utilizes the pressure variation in a pipeline caused by a change in flow
3. The Pitot tube method, in which a calibrated Pitot tube is used to traverse a penstock.

4. The current-meter method, which utilizes calibrated current meters for traversing the headrace, canal, or tailrace.

5. The weir method, which involves the construction of a weir whose coefficient of discharge is definitely known, usually in the tailrace. If the weir is left in place this method involves a permanent loss of head.

6. The Venturi meter, which has the advantage of providing a permanent record of discharge. A Venturi meter is sometimes installed in the penstocks of high-head plants, but this method involves a permanent loss of head for the purpose of measuring the water. The Venturi principle may, however, be utilized for securing a constant record of discharge without any additional loss of head [19]. Any contraction, such as inlet to casing or the speed ring, produces a loss of head. By properly placing piezometers and carrying them to recording instruments, such as are manufactured by Builders Iron Foundry, Providence, R. I., and Simplex Valve and Meter Company of Philadelphia, Pa., a permanent record of discharge may be obtained provided calibration is made originally by some suitable method.

The most advisable method of discharge measurement is determined by the particular conditions of the development, and, before reaching a decision as to the method to be used, the matter should be discussed with the turbine manufacturers.

All efficiency tests should be made in accordance with the Test Code for Hydraulic Prime Movers, 1938, or a later revision thereof, published by the American Society of Mechanical Engineers, New York City.

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CHAPTER 39

PARTS AND AUXILIARIES FOR HYDRAULIC TURBINES

By Arnold Pfau and the late Dr. William Monroe White

1. Introduction. The preceding chapter dealt with the various types of hydraulic turbines, their runners, setting, and draft tubes. This chapter deals with other parts of the turbine and its auxiliaries.

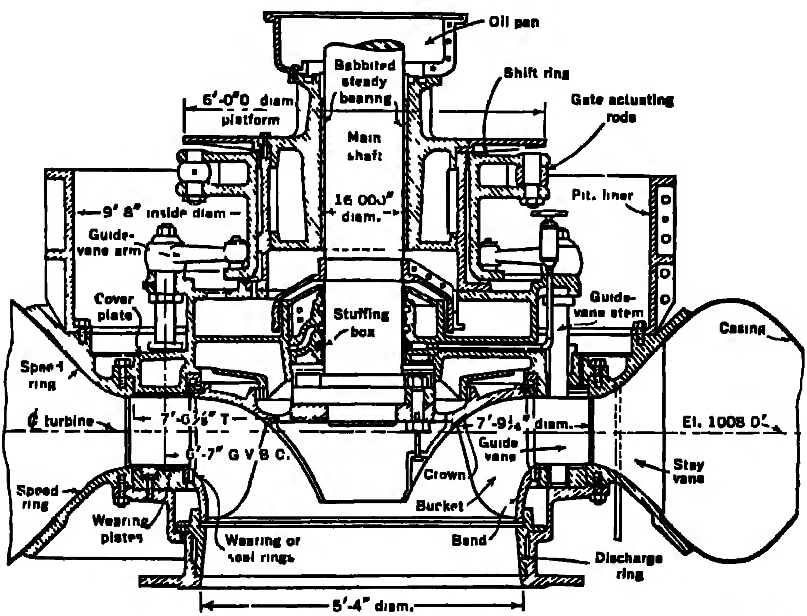


FIG. 1. Section through plate-steel spiral-cased turbine for 370-ft head, showing renewable discharge ring below runner, renewable wearing rings at runner clearances and wearing plates in guide case Bronze-bushed, individually lubricated guide-vane pivot bearings, one below water passage, two above with adjustable stuffing box between upper two bearings. Thrust bearing to take upward thrust on guide-vane stems. Cover plate of double construction drained through runner at center. Babbitted, oil-lubricated steady bearing with oil reservoir and viscosity pump in cover plate. Babbitted, grease-lubricated stuffing box with lignum vitae seal blocks and drains to prevent high tailwater entering oil reservoir. Shifting ring slides on bronze pads and connects to guide vanes through breaking links, eccentric adjusting pins, and long levers keyed to guide-vane stems. (Allis-Chalmers.)

The location of the various parts for Francis, adjustable-blade, and propeller types of turbines is shown in Figs. 1 and 2.

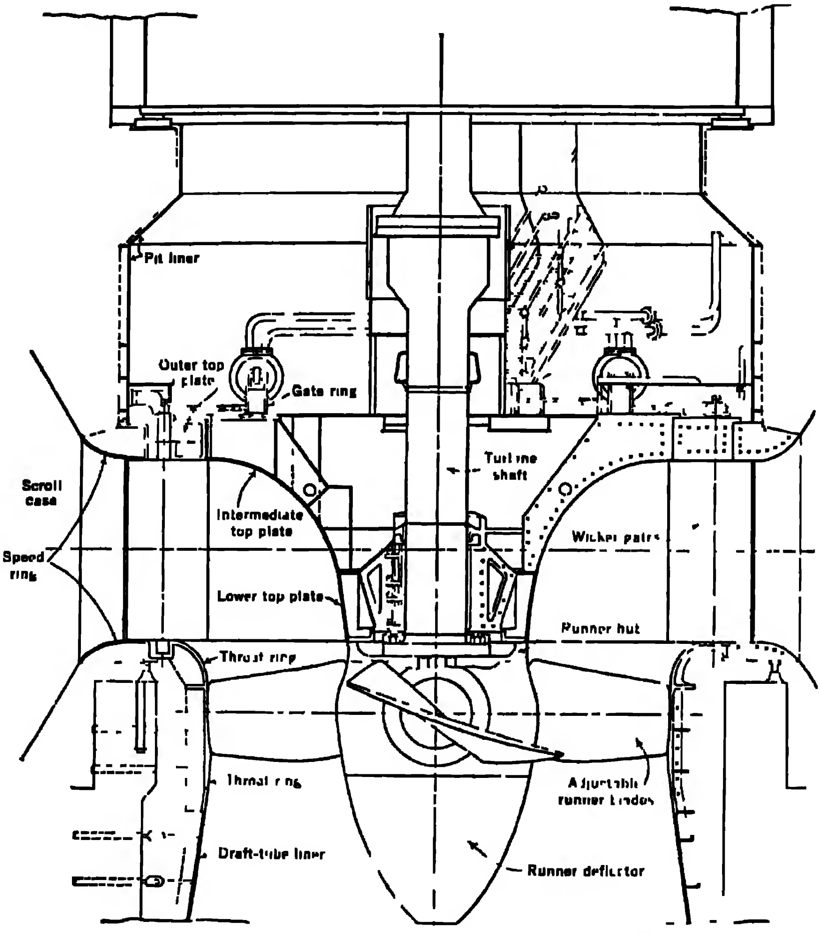


Fig. 2. Typical section adjustable-blade Kaplan turbine. (S. Morgan Smith.)

2. Speed Rings. The speed ring is that part of the turbine which connects the cover plate and the discharge ring. In its simplest construction it consists of upper and lower rings held together by vertical ribs, sometimes called stationary guide vanes or stay vanes. These ribs carry the load from above and take the force of the internal water pressure. There are three distinct types of speed rings: those for concrete spiral-cased units, those for plate-steel spiral-cased units, and those for cast-iron or cast-steel casings.

For cast-steel casings the general practice is to cast the speed ring and ribs integral with the cast-steel casing, as shown in Fig 3 This is desirable

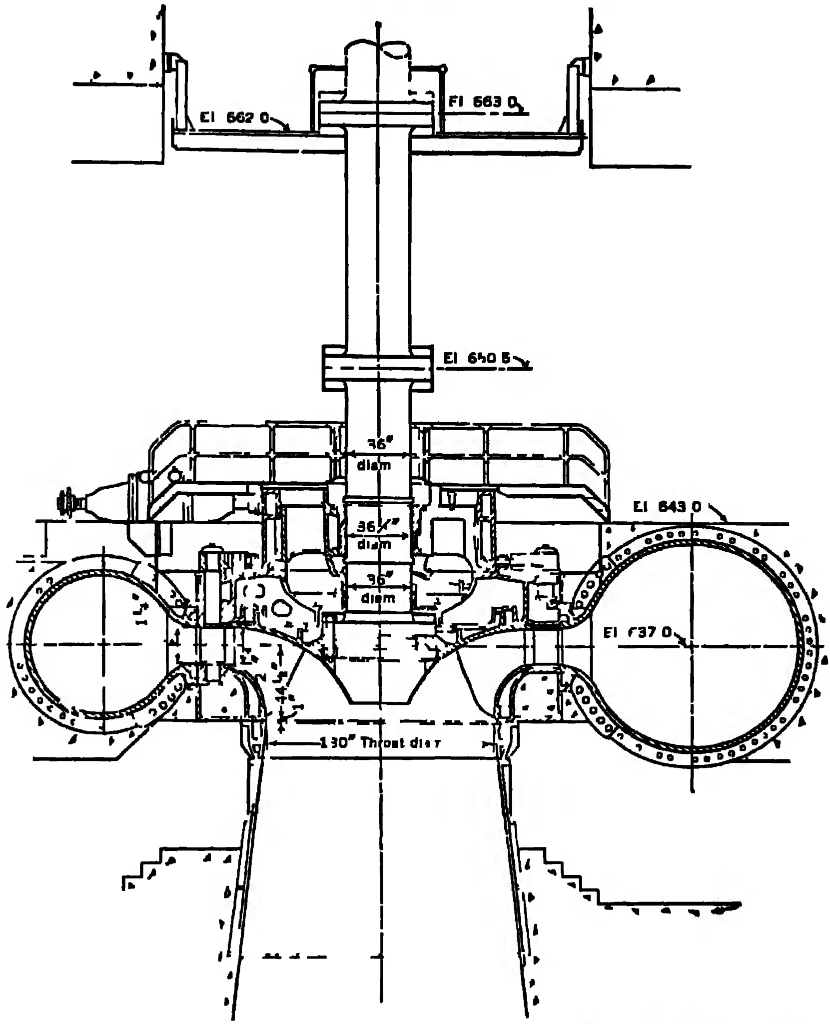


FIG 3 Combined cast-steel casing and speed ring for Hoover power plant 510-ft head 115 000 hp (I P Morris)

construction if the casting is properly made For medium and low head, with cast-iron- or cast-steel-cased units, the speed-ring ribs are sometimes made of turned bolts the casing simply is cast open on the inner diameter Because of the lower tensile strength of cast iron some trouble has been experienced in making the speed ring integral with the casing Conse-

quently, these features are generally cast separately for cast iron and bolted connections are used.

Speed rings for plate-steel spiral-cased turbines are usually of cast steel. The ribs and flanges are cast together, and the flanges extended and flared so that the plates may be riveted to them, as shown in Fig. 1. Cast iron is not considered suitable for this type of speed ring because it may crack in riveting, and although the cracks may not show up immediately they are likely to appear afterwards.

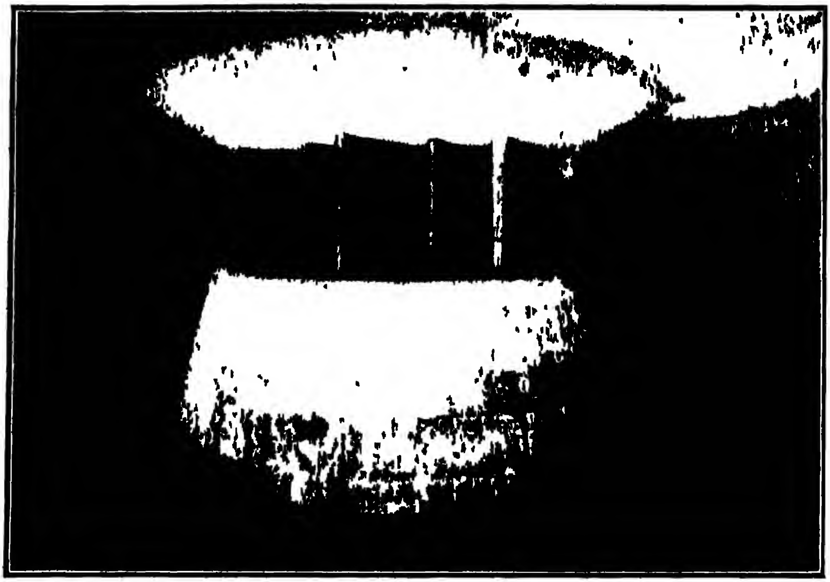


FIG. 4. View taken inside of concrete spiral-cased unit, 2500-hp, 21-ft head, 138½ rpm, showing cast-iron columns of speed ring and plate-steel guide vanes (Wausau Sulphate and Fibre Company) (Allis-Chalmers)

Speed rings for concrete spiral-cased units for low heads take a variety of forms, the most common of which are as follows:

Cast-iron speed rings, as shown in Fig. 8, Chapter 38, are very similar to those used with plate-steel casings, having the ribs and flanges cast together, and the reinforcing rods are sometimes attached to the flanges as the loads that they carry are very similar to those carried in the plate-steel-cased units. For large units under higher heads (say 70 ft), at which concrete scrolls may be used, cast steel may be required.

For small units the stay-bolt type of speed ring can be employed. In this construction the upper and lower rings are made of cast iron and the ribs are of steel rods. The rods are turned and threaded for bolting into the upper and lower flanges. In more desirable practice the steel rods are enveloped by stationary guide vanes of plate steel which help to carry the load.

The simplest construction of all, though for medium heads only, is to make only the ribs of cast iron, flaring them at the top and bottom and imbedding them in the concrete, relying entirely on the concrete for the upper and lower surfaces. With this construction, the ribs have no connection with the other turbine parts except where they are tied into the concrete. Under some conditions, only the lower ends of the ribs are imbedded in the concrete, as shown in Fig. 31, Chapter 38, and the upper ends are bolted to a built-up plate-steel flange.

Figure 4 is a photograph taken inside the concrete spiral casing for a vertical turbine showing the columns of the speed rings imbedded in concrete, which carry the superimposed weight.

The pit liner, cover plate, discharge ring, and guide-vane rings are generally bolted to the speed ring. The discharge ring in concrete-cased units is frequently omitted, as the lower flange of the speed ring serves this purpose. A plate-steel upper flange is usually designed to form the pit liner, when the cast ribs are used.

When the speed ring's diameter prevents the ring from being shipped in one piece, the sections are bolted or field-welded together, and sometimes additional shrink links are employed, especially for large plate-steel units.

3. Guide Vanes or Wicket Gates. The movable guide vanes, sometimes called wicket gates, properly control the flow into the runner. The guide-vane assembly, together with the speed ring, is sometimes called the distributor. The guide-vane mechanism for both the Francis and the propeller runners is of two general types: the open flume, or inside type, in which the connecting links are exposed to the flow of the water, and the incased or outside type, in which the upper guide-vane stems extend up through the cover plate and the operating mechanism is located on the cover plate outside the water. The open-flume guide vanes are usually made of cast iron, although some manufacturers make them of plate steel, the plates being formed and welded. The guide vanes for outside gate mechanism are generally constructed of cast steel. These have the one pivot extended through the cover plate, and, since this pivot must transmit the full torque to operate the guide vane, cast steel is desirable for strength.

Under modern operating conditions it is often necessary to shut the unit down with the head gates open and the case full of water so that the unit may be quickly placed on the line if needed. Full water pressure is thus put on the closed guide vanes or wicket gates, possibly for a long time. If the gates are weak, out of alinement, or poorly assembled, considerable leakage loss will occur. This leakage may be of material economic importance in times of low flow.

Unfortunately, there is sometimes no way of readily adjusting the guide vanes to stop this leakage definitely. The authors have known such leakage to amount to 25% of full plant discharge, and even for well-operated and well-maintained plants, it has exceeded 10% of minimum stream flow. It is entirely feasible, however, to reduce it to a point where it will be of no great

economic importance by means of more precisely machined surfaces with closer clearances, improved design of link and operating mechanism, better sealing devices, and more careful maintenance. In the opinion of the authors, more emphasis should be placed on the desirability of tight wicket gates, and a considerably higher cost than usual would often be justified to secure greater watertightness.

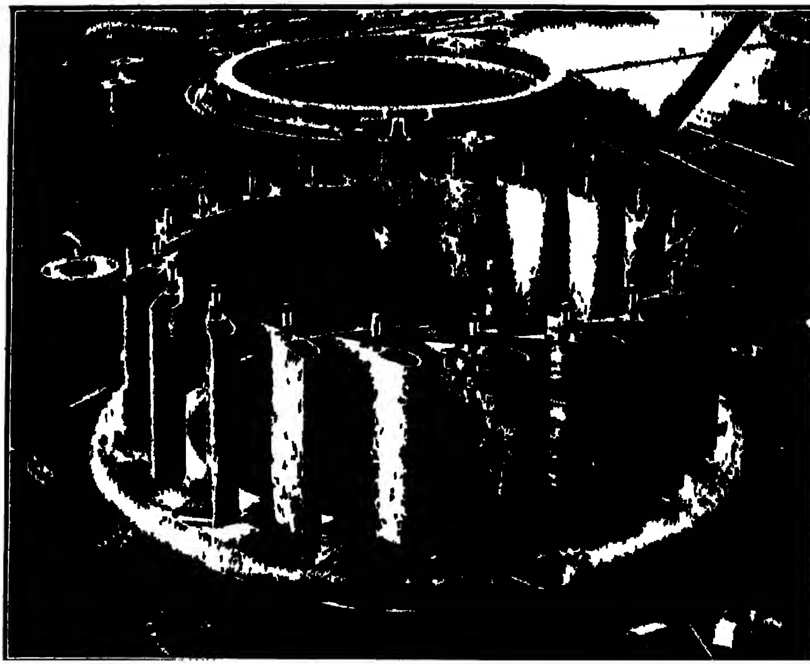


FIG. 5. Guide vanes and mechanism for open-flume turbine Guide vanes turn on bolts screwed into discharge ring, and are operated by links connecting to shifting ring, which rests on discharge ring Guide vanes shown about 50% open

For medium- and high-head plants it is often desirable to install a tight valve of the butterfly-gate or needle-valve type (Chapter 26) in the penstock just ahead of the scroll case to avoid leakage losses during shutdown periods. The cylinder gate at the turbine has also been used to eliminate this leakage, as at Rocky River, Connecticut. Such valves or gates often pay handsome returns on their cost in water saving.

Figure 5 shows the arrangement of guide vanes for an open-flume turbine. The guide-vane bolts or pivots are screwed into the discharge ring at the bottom, and the upper end extends up through the cover plate, which has been removed from this photograph. These guide vanes are of cast iron with bronze bushing pressed in to form bearings. Guide vanes or wicket gates are now also built up by constructing sections of plate around dies, welding

the edge where the plate ends meet, and welding steel blocks in to form the bearings. Figure 5 shows the arrangement of links and the movable shifting ring located on the discharge ring for moving the guide vanes. This mechanism is known as "inside type" because the links and pivots are inside the wheel pit and exposed to the water.

The stationary guide-vane bolts should be designed so that each guide vane can be removed without requiring removal of any adjacent part.

The customary guide-vane operating mechanism for the open-flume turbine consists of a shifting ring located either above or below the guide vanes and mounted on pads so that it can rotate through a small angle. Connection is provided by two adjustable rods from lugs on the shifting ring to a lever on an operating shaft which connects to the governor. Short links are not made with a breaking section. A refinement in this design is to make one of the pins slightly eccentric so that some adjustment can be made and the gates closed uniformly to decrease leakage.

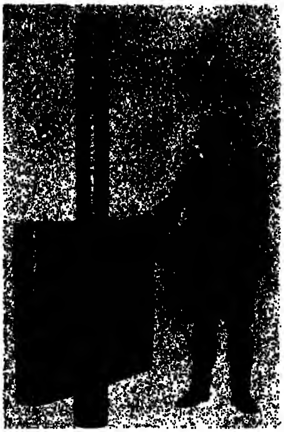


FIG. 6. Cast-steel guide vane for turbine having outside gate mechanism. Upper stem extends through cover plate and connects to shifting ring through lever and link. For 70,000-hp Niagara Falls turbine. (Allis-Chalmers.)

A similar arrangement can be used with the outside gate mechanism, especially for medium- and high-head turbines, where leakage during shutdown periods may cause considerable damage to the turbine parts. If some obstruction clogs the vanes, even though they are provided with breaking links, there is likely to be some distortion of the guide-vane lever or stem. To obtain a tight closing fit, if the turbine is not equipped with eccentric adjusting pins, it would be necessary to fit the levers to the guide vanes with offset keys—a procedure that means a long shutdown.

An alternative for breaking links is an element that is accessible above the cover plate and that fails by shear when débris prevents movement of the guide vane. It is essential that with either design care be taken to avoid mechanical interference when the remaining guide vanes move on.

Figure 6 shows a large cast-steel guide vane of the outside-gate type, such as is used with cast, steel-plate, and concrete spiral-cased turbines. The turbine shown in Fig. 34, Chapter 38, employs such a guide vane. The lower pivot is carried in a bronze bushing in the lower guide-vane ring, which is lubricated from the top by grease forced through a hole bored through the stem. The upper pivot extends through two bearings in the cover plate, and an adjustable packing box is provided between these bearings to prevent leakage along the stems. A lever is keyed to the pivot above the upper

bearing and is connected to the shifting ring with short links that are designed to break if some obstruction clogs the gates. The connection of levers, links, and shifting ring is shown in Fig. 7, which is a shop assembly of one of the Hoover Dam units.

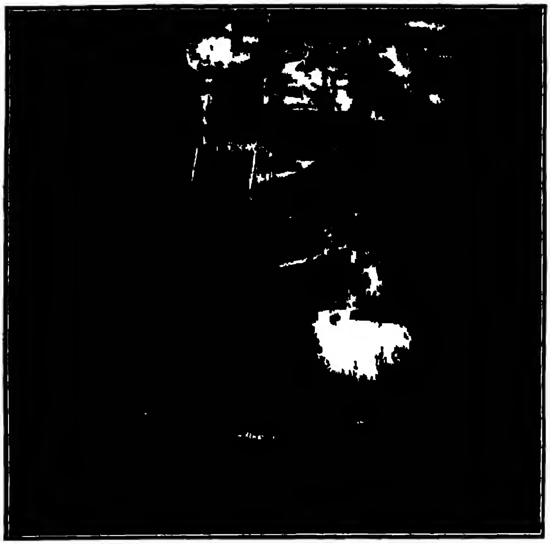


FIG. 7. Shop view of 115,000-hp, 510-ft head, 180-rpm Hoover Dam unit (I. P. Morris.)

4. Wearing Rings and Facing Plates. Wearing rings and facing plates above and below the guide vanes have renewable surfaces. They are provided where it is desirable to maintain small clearances in order that leakage may be held down to the minimum Facing plates serve also to line the two surfaces of the casing adjacent to the ends of the guide vanes (see Fig. 1) They are usually constructed of steel or bronze from 3/8 to 3/4 in thick, made either in sections or in continuous rings, and are used on most large turbines for heads above 75 ft. When the turbine is shut down and the guide vanes are closed, the water tends to leak past the end of the guide vanes, and, if it contains sand or foreign matter, it quickly wears away the surfaces at this point. As higher heads and larger-capacity turbines are used, it is important to decrease this leakage, and the machines should be designed so that they can be readily restored to their original condition.

Wearing rings are employed on the clearances adjacent to the runner band and crown. They are shown distinctly in Fig 1. Sand and foreign matter in the water rapidly wear these clearances, especially at heads above 200 ft. With runners below 30 specific speed (heads greater than 400 ft), the leakage through these clearances during operation, even in their original condition, sometimes amounts to as much as 3 or 4%, which, of course, represents a corresponding loss in efficiency. Any wear will greatly increase

this loss. Labyrinth seals at the top and bottom of the runner reduce this leakage about in proportion to the number of stages. They are formed by several stages of wearing rings.

It has become common practice to omit the wearing rings on the runner band but to provide them on the stationary surface of the discharge ring and cover rings since these may be more readily renewed. Experience has shown that only in a few cases have the wearing rings been renewed on the runner.

When wearing rings are used both on stationary parts and on the runner, the material in the wearing rings on the stationary part is generally of lower Brinell hardness than the wearing rings on the runner.

Provision should be made for periodically checking the running clearances.

5. Regulating Connections. These comprise the mechanism connecting the guide vanes to the governor. The guide vanes, described in Section 3, are attached to a common shifting ring sliding on machined ways provided on the cover plate of the turbine, as in Fig. 7.

In the open-flume setting an actuating shaft operated from above by the governor servomotor is linked directly to the shifting ring.

For some spiral-casing units of medium size, such as shown in Fig. 32, Chapter 38, the same types of regulating connections as described above are used even though the guide-vane mechanism is of the outside type. It has been found that, for units requiring a governor not exceeding 25,000- or 30,000-ft-lb capacity, the regulating shaft mechanism described above is the most economical, but when the required governor capacity exceeds approximately 30,000 ft-lb, it is usually less expensive to locate the regulating cylinders in the turbine pit and connect one piston rod each side of the shifting ring. The customary construction is shown in Fig. 8 and Fig. 37, Chapter 38. This arrangement makes a positive and direct connection, practically equal forces being exerted on both sides of the shifting-ring surface. Provision must be made for supporting these regulating cylinders rigidly.

6. Main Shaft. The main shaft of the hydraulic turbine must transmit the full torque of the runner under the maximum head conditions at which the turbine will operate. In addition, it must carry in tension the weight of the runner and the hydraulic thrust. If the main shaft is designed for a normal full-load stress under torsion alone of approximately 4000 lb per sq in., there is a sufficient margin of strength to carry the additional thrust loads.

The approximate diameter of shaft required for a torsion stress of 4000 lb per sq in. can be computed from the following formula:

$$4.5\left(\frac{P}{N}\right)^{1/3} \qquad [1]$$

where d = shaft diameter, in inches;

P = horsepower;

N = revolutions per minute.

The main shaft must be sufficiently rigid so that it will not vibrate or whip. Runners become clogged with débris; if obstruction occurs on one side of the runner only, the unit may be thrown out of balance. To safeguard against serious vibration in this event, the main shaft should have an ample margin of strength to give stiffness.

Critical Speed of Shaft. If small shafts under high speed are employed, or if the runner is overhung at some distance from the bearing, the matter of critical speed under runaway conditions should be carefully investigated. If necessary, the shaft should be increased in diameter or an additional steady bearing provided so that, under maximum runaway conditions, the critical speed for the largest span will be not less than twice the runaway. The following formula for critical speed can be used when only the weight of the shaft and no additional bending forces is considered:

$$N = \frac{30{,}700d}{L^2} \qquad [2]$$

where N = critical speed, in revolutions per minute;
d = shaft diameter, in inches;
L = span between bearings, in feet.

It has been found that forged open-hearth steel shafts of approximately the chemical and physical properties tabulated give excellent results.

PHYSICAL

Ultimate strength	60,000–75,000 lb per sq in.
Elastic limit	30,000–42,000 lb per sq in.
Elongation in 2 in.	About 25%
Reduction in area	About 35%

CHEMICAL

Carbon	0.25%–0.35%
Sulphur, not over	0.045%
Phosphorus, not over	0.045%
Silicon	0.02%
Manganese	0.40%–0.60%

While high-carbon-steel shafts and special chrome steels may have greater physical characteristics, they have not been found entirely satisfactory under the conditions to which they are exposed. Hydraulic turbine shafts are frequently subjected to shock caused by unstable conditions, and for this reason a fairly soft, ductile material is advisable. Large shafts should be hollow-bored and inspected for flaws. All important shafts should have test bars and specimens taken from both ends. Hollow-forged shafts are usually of more uniform material, but their excessive cost has not warranted their general use.

There are several different methods of connection between turbine shaft and turbine, and turbine and generator shafts. The shafts on the small

units of the early days were usually of cold-rolled steel, and separate flanges were pressed and keyed onto the end. The runner was generally pressed on a straight fit on the lower end of the shaft, where it was keyed and locked. There are two approved methods of attaching runners to shafts. The more common is a taper fit, in which the runner is pressed onto the tapered end of the main shaft, where it is securely keyed and then prevented from moving back on the taper by a split-stop ring that fits into an annular groove on the main shaft.

FIG. 8. Turbine shafts with forged flange at both ends. Largest 34-in. diameter, 6-in. hollow bored, split-steel sleeve where shaft passes through bearing.

Figure 8 shows water-wheel shafts with forged flanges at both ends—one end for coupling to the generator flange, the other for bolting to the runner. The larger shaft shown has a steel sleeve around it which is split and is held around the shaft by shrink links. A key, similar to that shown on the small shaft in the foreground, prevents the sleeve from turning on the shaft.

Figure 9 shows the method of fitting keys where the runner is pressed onto a tapered main shaft. Here, each slot has two keys, which are tapered

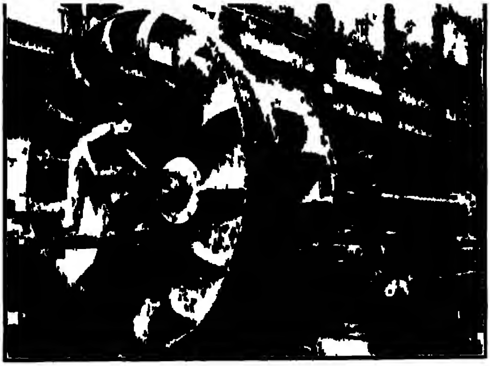


FIG. 9. Fitting key on 170-in. discharge diameter runner, rated 24,000 hp at 70-ft head. Runner is pressed onto tapered end of main shaft, key is inserted, then stop ring is inserted in groove in end of shaft to prevent endwise motion. Cap incloses end of shaft and holds stop ring in place. (Allis-Chalmers.)

to provide a wedging action. After the key has been driven, the stop ring is fitted into the shaft shown just below the end of the runner fit, and then a cone or tip is bolted to the runner, inclosing the end of the shaft and preventing the stop ring from working out.

When the main turbine guide bearings are of the babbitted oil-lubricated type, the main shaft is allowed to come in direct contact with the bearing surface; but when the water-lubricated main bearings are used it is excellent practice to provide either a stainless-steel or bronze sleeve around the shaft in order to take the wear. This enables replacement to be made without weakening the shaft. Considerable wear is likely to occur in water-lubricated bearings, especially if any sediment or sand is in the water. In oil-lubricated babbitt-type bearings, however, there is very little danger of wear on the shaft.

For the sleeve, steel has been found a much better material than bronze as it will stand the rubbing action and show much less wear. These sleeves can be made in one piece and shrunk on, or they can be split axially and held on with shrink links. A less desirable means of fastening the split sleeve to the shaft is countersunk screws tapped into the shaft. Such sleeves are difficult to replace properly, because it is not easy to match the old taps in the shaft, and to drill new ones repeatedly is bound to weaken the shaft

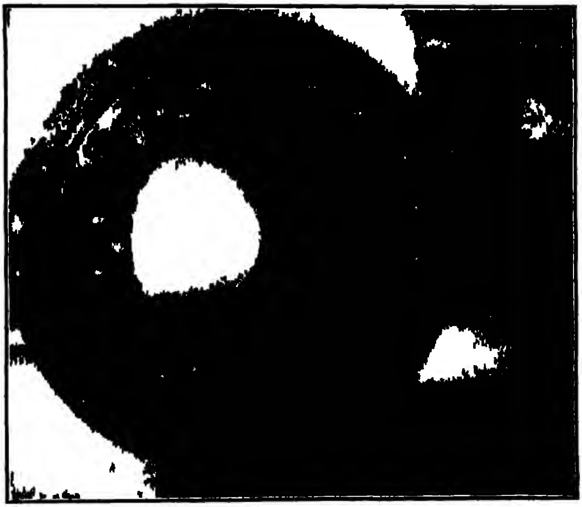
7. Guide Bearings. Turbine guide bearings for vertical units are of two principal types. Those for water lubrication (Fig. 10) are usually of lignum vitae or a similar wood or fibrous material, and those for oil lubrication (Fig. 1) are of the babbitted type. It will be noted in Fig 10 that there are six adjustable shoes, each of which is lined with phenolic strips, in this case in lieu of lignum vitae blocks. The shoes are guided in a tapered seat in the bearing housing so that adjustment for wear is provided by lowering the six shoes, thus reducing the radial clearance.

Water-lubricated bearings are used on the majority of installations, both the open-flume type and the inclosed spiral-cased turbines of the vertical type. Where clean water is available, or where installation of a settling tank is possible, bearings of this sort last many years without renewal; but where there is grit in the water or where during flood times only dirty water is available, they become worn quickly, sometimes allowing the runner to rub. On the small open-flume turbines, the bearings are usually constructed simply by setting four blocks in pockets cast in an iron bearing shell and by the provision of adjusting screws to hold the blocks against the shaft. In the large size, a number of strips of the bearing material are set into dovetail grooves cast into the bearing shell. These bearings are not adjustable. When worn, they must be relined and rebored. However, bearings of this type have been developed in which sections or shoes of cast iron containing the dovetailed grooves are set in an outer housing so that they can be adjusted when the bearing wears.

When the water-lubricated bearing is operated submerged, as in the open-flume construction, no other lubrication is required, although sometimes grease is used in addition This applies both to vertical-shaft and to horizontal-shaft units. In the incased units, a steady stream of water to the bearing is provided, and grease is often supplied also. Some type of strainer for the water supply is advisable, and it should be in duplicate so that one

strainer can be cleaned while the other one is in use Duplicate strainers are desirable with this type of bearing, especially in cold climates where there is

(a)



(b)

FIG 10 Water-lubricated guide bearing, 36 000-hp, 36-ft head, 75 rpm, turbine, Chickamauga plant, T V A, shaft diameter 37 in (a) Housing bearing, (b) assembled bearing with shoes and bearing strips in place (Courtesy I P Morris Department, Baldwin Locomotive Works)

danger of one strainer being clogged with fine ice, allowing the bearings to run dry In some plants this has been remedied by running the water through a moving screen, and in others the water is run through warming tanks before it enters the bearing

Considerable success has attended the substitution of "Insurok," "Rycrtex," and other plastic fibrous compounds for lignum vitae blocks in water-lubricated bearings. Rubber bearings have also been successful. Longer wear is claimed for these materials.

Oil-lubricated bearings of the horizontal type, as shown in Figs. 40 and 44, Chapter 38, have long been in general use. A considerable number of installations of the vertical-shaft units with oil-lubricated turbine bearings as shown in Fig. 1 are operating very satisfactorily. Some of these have a reservoir just below the bearing, located in the cover plate, and one or two pumps driven from the shaft, which lift the oil to the top and allow it to return through the bearing. In other installations, the oil is circulated through an external tank by a motor-driven pump. The tank system, however, is not so compact or so reliable as the self-contained system.

Probably the reason why vertical oil-lubricated bearings are not more popular is that some engineers fear that the oil will be lost on account of the suction along the shaft. With the proper design this will not occur. If a good oil slinger is provided below the bearing, as well as a suitable stuffing box with ample drain area, there is no danger of losing oil. Bearings have been in operation for several years without appreciable loss of oil. In the event of high tailwater, there is some danger of the bearing's being flooded out when the unit is shut down, but this hazard also can be prevented by a suitable packing box and drain to a sump. If the unit is in operation, the vacuum below the runner is usually sufficient to prevent any rise of water along the shaft.

Horizontal bearings are usually of the ring-oiled pedestal type, similar to those on generators, and have proved very satisfactory. Generally the circulation of oil from the rings is ample, although some large bearings have been designed to use oil under pressure when starting up or even when operating.

The present trend in oil-lubricated guide bearings, especially for large units such as Hoover, Grand Coulee, etc., is to use the adjustable-shaft packing box rather than the labyrinth seal type. Since the adjustable type requires frequent inspection and adjustment, it is essential that it be readily accessible.

8. Thrust Bearings.* The thrust bearing, usually mounted on the upper or lower bridge of the generator, supports a thrust load amounting to several hundred tons on a large vertical unit. The thrust load consists of the weight of the rotating parts and the hydraulic thrust of the water on the buckets or blades of the runner. The hydraulic thrust can be calculated from the following equation:

$$T = \frac{KD^2H}{2.94} \qquad [3]$$

* Data furnished by A. B. Lakey, Chief Engineer, Kingsbury Machine Works, Inc.

where T = hydraulic thrust, in pounds;
D = discharge diameter of runner, in inches;
H = head on turbine, in feet (use maximum prevailing head);
K = thrust coefficient as given in Fig. 11. (For fixed- and adjustable-blade propeller runners, K = 1.0.)

To obtain the total thrust the weight of the revolving parts must be added to the hydraulic thrust obtained from the formula.

In order that the bearings will carry the required loads without undue wear and friction, they are designed so that a film of oil is maintained be-

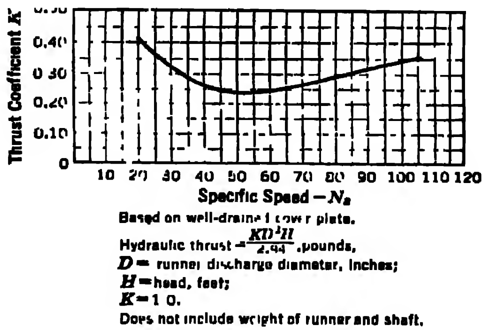


FIG. 11. Hydraulic thrust chart for fixed- and adjustable-blade propeller runner

tween the running and stationary bearing surfaces. The running surface, or thrust collar, is smoothly ground iron or steel; the stationary surface is usually babbitted. One of the surfaces—either the moving or the stationary, depending on the design of the bearing—is divided by ample grooves into six or eight pie-shaped sectors. Each of these sectors is tilted very slightly (see Fig. 12) so that oil from the surrounding bath is carried by viscous drag between the two bearing surfaces to form a wedge-shaped film. Experience has shown that the oil pressure within the wedge varies parabolically from zero at the four edges of the sector to maximum near the center.

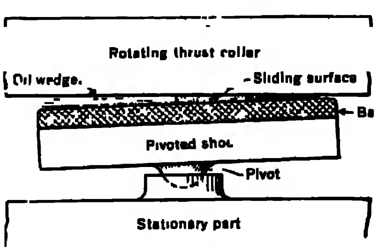


FIG. 12. Diagrammatic sketch of Kingsbury (pivoted shoe) thrust bearing. Tilt of shoe exaggerated.

The Gibbs, or fixed-taper, type of thrust bearing has slightly beveled sectors formed in the revolving member. The amount of bevel is selected with reference to the expected surface speed, load, and oil viscosity. Each taper begins at a radial groove and ends in a parallel-surfaced land upon which the bearing rests when not running.

The Reist, or spring-supported, type of thrust bearing has a flat, babbitted, stationary plate with a number of radial grooves. The plate is relatively thin and flexible and is supported uniformly by numerous springs, as indicated in Fig 13. It is obvious that any variation in total load changes the deflection of these springs, whereby the rotating element assumes a dif-

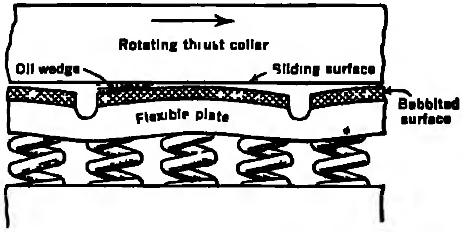


FIG 13 Diagrammatic sketch of Reist spring-supported type of thrust bearing Deflection of plate exaggerated

ferent setting with reference to the center plan of the runner inlet This difference may be very appreciable between the extreme conditions of operation The surface is thus capable of adjusting itself to a favorable wedge-shaped film form

The Kingsbury, or pivoted segment, thrust bearing has stationary, flat, babbitted shoes, as indicated in Fig 12 The segments or shoes are sup-

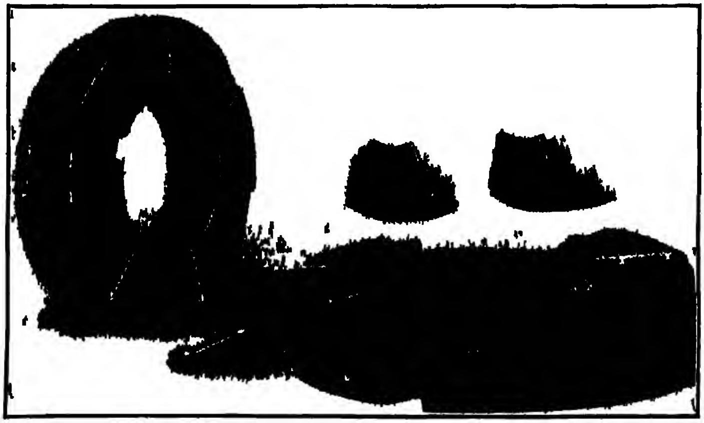


FIG 14 Two views of stationary parts, Kingsbury thrust bearing.

ported at the centers of gravity on a rounded pivot so that they can tilt to the proper angle to give the correct wedge-shaped oil film for the given conditions of load Figure 14 shows two views of the stationary part of a Kingsbury bearing with six pivoted shoes The bottom and top views of a single shoe are shown also

The thrust runner or collar that bears on the pivoted shoes is a flat disk fastened to or turned integrally on the shaft. The thrust runner surface may be polished, especially if the turbine is of the adjustable-blade type that in starting has a small torque and the maximum hydraulic thrust load.

The load ratings of the bearings are based on maintaining a definite oil thickness at the thin edge of the film for a given speed and maximum thrust load; the thickness increases with the size of the bearing. Normally the oil used has a viscosity of about 130 Saybolt Universal at the mean bath temperature. Unit bearing pressures at this viscosity range from 350 lb per sq in. for medium sizes up to 600 lb per sq in. on large, fast-moving bearings.

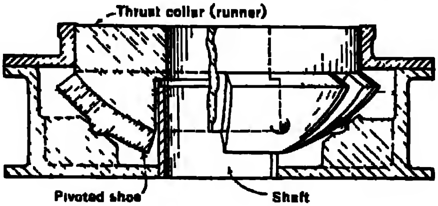


FIG. 15 Diagrammatic sketch of Kingsbury spherical thrust bearing

Kingsbury spherical thrust bearings are in service on a number of units. This type of bearing has a thrust runner in the form of a zone of a sphere so that it combines the features of a thrust and a radial guide bearing and is self-aligning. The stationary part of the bearing consists of six concave shoes mounted on pivots, as indicated in Fig. 15. The spherical seat is a sufficient guide for side forces up to 75% of the downward load. The largest bearing of this type which is in service is 51 in. O.D., supporting a thrust load of 555,000 lb running at a speed of 450 rpm. The thrust load rating for the spherical bearing is about the same as for the flat Kingsbury bearing of the same diameter.

The approximate size, outside diameter of shoes, and other characteristics for a flat Kingsbury thrust bearing can be estimated from the chart in Fig 16, which shows data for both Francis and propeller runners of normal proportions. The chart assumes the most usual bearing, which contains six shoes subtending arcs of 51 degrees, the bore of the shoe being about 43% of the outer bearing size. The net bearing area is thus $0.55B^2$, where B is the outer bearing diameter in inches. In special cases the bore ratio may be as great as 56% of the bearing diameter. Nantahala (see Table 1, Chapter 38) is an example of a large-capacity, high-head Francis runner supported by a Kingsbury spherical thrust bearing that has operated very satisfactorily. Figure 17 is a section through the Nantahala unit showing the spherical thrust bearing above the generator.

A cooling coil immersed in the oil bath is used with the largest and highest-speed thrust bearings and is generally designed to limit the oil temperature to

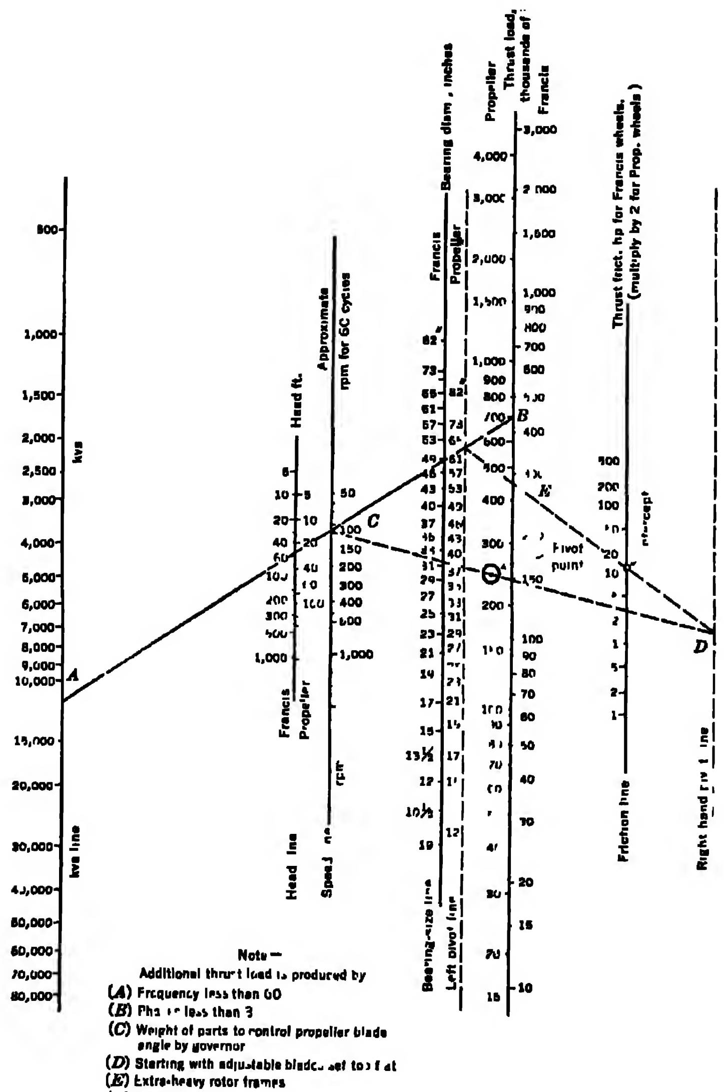


FIG. 16. Chart for estimating thrust loads and bearing sizes for Kingsbury thrust bearings for vertical hydroelectric units (Kingsbury Machine Works, Philadelphia, Pa)

about 45 to 50 degrees centigrade (113 to 122 degrees Fahrenheit). Usually a flow of 1 gpm of water is allowed for a friction loss of 1 hp, which corresponds to a rise in water temperature of about 5 degrees Fahrenheit. Flow rates are sometimes as low as ⅔ gpm, increasing the water temperature rise to about 7.5 degrees Fahrenheit.

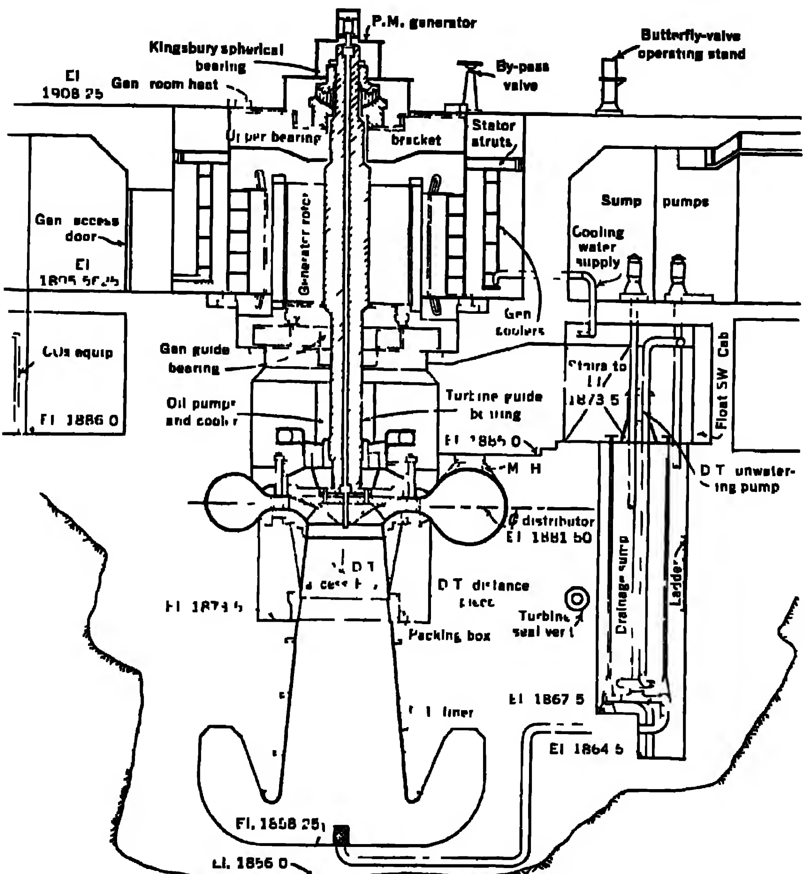


FIG. 17. Nantahala plant, North Carolina. Section through 60,000-hp, 920-ft head Francis turbine (Newport News). Note spherical thrust bearing.

Example Worked on Chart, Fig. 16.

Given: unit with propeller turbine; 12,000 kva, 3-phase, 60-cycle, 25-ft normal head.

Required: thrust load, bearing size, friction loss in bearing.

Solution: Line AB is drawn from 12,000 kva on kva line through 25 ft on propeller side of head line, extending to intersect thrust line. Intersec-

tion with speed line indicates 100 rpm. Intersection with bearing-size line shows bearing diameter should be 61 in. for propeller wheel. Intersection with thrust line at B indicates total thrust load is 700,000 lb.

For friction loss, line CD is drawn from intersection of line AB with speed line passing through pivot point O to intersect right-hand pivot line at D. Line DE is drawn connecting point D with intersection of line AB on left-hand pivot line between bearing and thrust lines. Point where line DE crosses friction line indicates that friction power loss in bearing is 13 hp for a Francis runner, or 26 hp for a propeller wheel. Cooling water required is 26×1 or 26 gpm for a 5-degree-Fahrenheit temperature rise.

9. Cover Plates. Cover plates on open-flume turbines are usually made of cast iron and can be very simple in construction as long as they meet the principal requirement, sufficient rigidity to hold the proper alinement of the guide vanes. Sometimes stay bolts from the discharge ring serve to add rigidity. Another method is to use struts or braces from the flume walls, as shown in Fig. 25, Chapter 38. Some such support is necessary, because the water passing through the guide vanes exerts a force upon them. The discharge ring of the turbine, being attached directly to the foundations, can carry its share of the load quite readily; but unless the cover plate is supported rigidly it will tend to rotate slightly, setting up vibrations that have been known to break off the guide-vane stems. Consequently, the more satisfactory support is from the flume walls rather than with stay bolts.

A better and more expensive design provides a complete cast speed ring (see Section 2), to the top of which the cover plate is firmly bolted.

Cover plates of concrete spiral-cased and plate-steel-cased turbines are more elaborately constructed. The bearings for the guide-vane stems, where they extend through the cover plates, should be bronze-bushed, and some provision should be made for carrying away whatever water leaks up along the guide-vane stems. Sometimes the cover plate is designed so that any leakage water through the lower bearing, if water-lubricated, is drained over and down into the draft tube through the cored openings in the runner crown.

The cover plate shown in Fig. 1 is of a more elaborate design suitable for large units. It has two bearings above the guide vanes, and between these two bearings an adjustable packing box is provided so that all leakage along the bearing can be eliminated. This cover plate and runner are designed with labyrinth seals to decrease the leakage past the runner clearance. There is also a cored passage in the cover plate to lead this water over to the drain holes in the center of the runner. This is known as a "double-deck" cover plate. It prevents the building up of pressure on the runner crown, thus reducing the load on the thrust bearing.

Figure 18 shows a comparison of two types of guide-vane bearing and stuffing box on the cover plate. The one at the left with adjustable packing box between two bearings is the more substantial construction. The design

shown on the right of Fig. 18 does not prove satisfactory if the packing is ordinary hemp material, which does not act automatically under pressure. More recent packing materials now available, such as Chevron, prove very satisfactory, being automatically self-adjusting.

Figure 2 shows a typical cover plate for a propeller type of turbine. As may be seen from the figure, the cover plate acts as a guide for the water from the gates to the runner and is therefore much deeper than cover plates for Francis runners. Owing to this depth, it is possible to locate the lower guide bearing and seal below the level of the gate tops, thus saving space above the turbine.

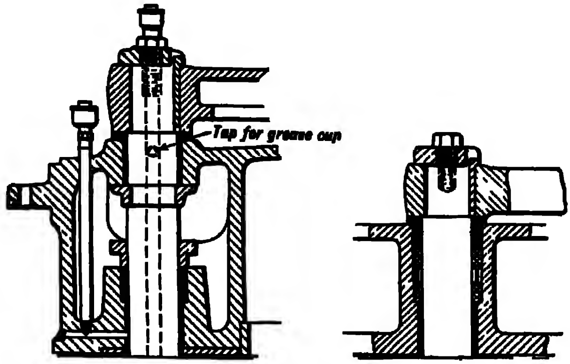


FIG. 18. Alternative types of guide-vane and cover-plate design, the one at the left having two bearings with adjustable packing between, the other having packing forced in between two bushings.

10. Turbine Venting. Venting of turbine discharge is often desirable or even necessary. For both high- and low-head turbines, the admission of air under atmospheric pressure at partial gate openings may materially reduce vibration and cavitation without impairing efficiency. Usually so-called turbovents are mounted in the turbine cover plate and are connected with the movable guide-vane mechanism so that when guide vanes are at the part-open position, which would otherwise cause vibration, the turbovent automatically opens and admits air at atmospheric pressure.

The admission of air is often desirable when the unit is operating as a synchronous condenser with wicket gates closed. If the water in the draft tube is low enough so that the runner revolves in air, instead of churning the water, the losses are greatly reduced. For instance, in one case the input of power to a unit operating as a synchronous condenser was reduced from 500 kw to 50 kw by the introduction of compressed air. Care must be taken to lubricate the seal rings to prevent heating and seizing.

11. Governors. Hydraulic turbine governors are designed to regulate the speed of the unit within a desired range by increasing or decreasing the amount of water supplied to the turbine runner in order to maintain a balance between power input and power demand, acting when a change in

power demand causes a fluctuation in speed. This balance is accomplished by the following principal elements of the governor:

1. The speed-responsive element, commonly termed flyballs or speed governor (see Fig. 19).

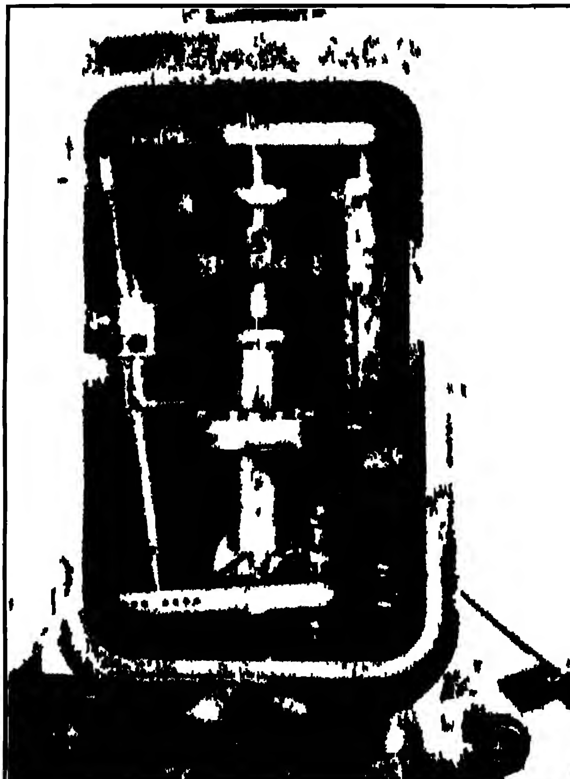


FIG. 19. Governor actuator with supersensitive motor-driven high-speed flyballs, pilot servomotor relay, and compensating dashpot. (Allis-Chalmers pat. design.)

2. The control valve to supply fluid under pressure to the servomotor in order to actuate the turbine control mechanism.
3. The servomotor, a fluid-pressure-operated piston or pistons which move the turbine control mechanism (Fig. 20).
4. The restoring or relay mechanism to hold the servomotor in the correct fixed (or dead-beat) position when the turbine output and load demand are equalized. (It can be direct mechanical drive or electrical.)
5. The fluid-pressure supply required for the action of the servomotor.

The essential action of the governor when a change in load occurs is shown diagrammatically by Fig. 20. Here the flyballs are driven by an electric motor receiving current from a separate generator operated in synchronism

with the turbine. The full lines show the position of the various parts of the governor when the unit is running at constant speed under constant load. The servomotor piston is shown at about mid-position, indicating that the wheel gates are about half open.

When additional load is suddenly thrown onto the unit, the wheel shaft slows down slightly, causing the governor drive (or pilot generator and

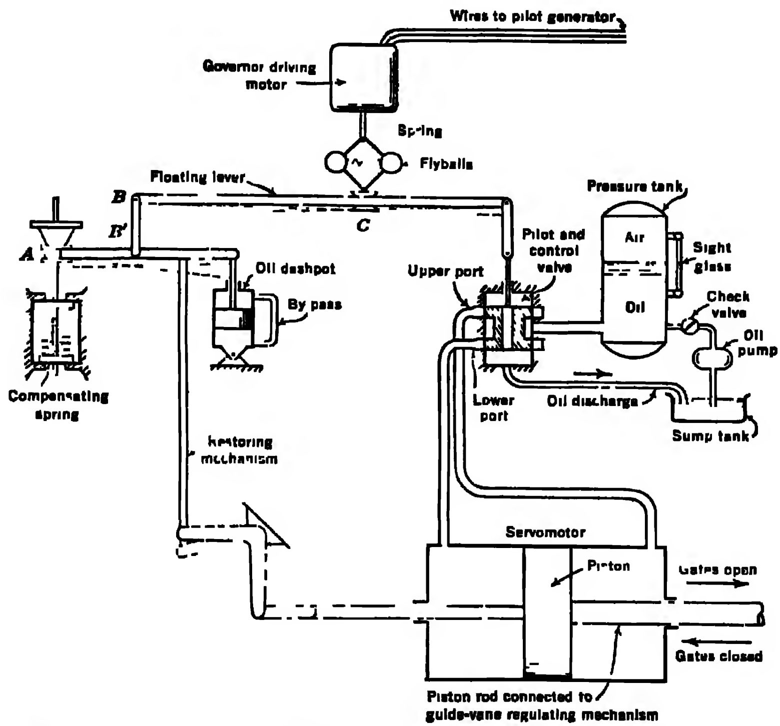


FIG. 20. Diagrammatic sketch showing operation of governor. Pilot and control valve here shown to be the same, whereas actually for sensitive governor small pilot valve moved by flyballs actuates larger control, or relay, valve hydraulically, as shown in Fig. 23.

governor driving motor) to reduce speed also. The flyball centrifugal force is reduced so that the spring pulls them inward, causing the floating lever to drop and displace the control valve downward from its central, or dead-beat, position. During this initial movement the far end *B* of the floating lever is still held in its original position by the servomotor connection.

When the spool-shaped control valve is moved downward, the lower port is uncovered so that oil flows from the pressure tank around the spool and through the port into the left-hand end of the servomotor cylinder, causing the piston to move to the right and open the gates. The top port

of the control valve is also uncovered so that oil in the right-hand end of the servomotor flows out and down through the center of the spool valve, discharging into the sump tank.

The movement of the piston rod displaces the restoring mechanism that moves the dashpot lever downward about the fulcrum A. The left end of the floating lever is lowered from B to B', causing the right end to lift about the fulcrum C, bringing the control valve back to neutral and thus stopping further movement of the gates. As the speed returns to normal, the flyballs gradually return the floating lever to its normal position. If the piston movement is too rapid, the oil in the dashpot cannot by-pass fast enough and the fulcrum A moves against an adjustable spring that brings the control valve more quickly to the neutral position, tending to prevent overtravel. If there were no dashpot and restoring mechanism to delay the action, the governor would "hunt" and the gates would be constantly changing.

When load is rejected, the operation of the governor is the reverse of that just described, the initial movement of the flyballs being outward because of an increase in speed and the control valve being lifted upward, admitting oil under pressure to the upper port shown in Fig. 20.

As the governor is operated and oil from the pressure tank is used, the air pressure is reduced so that a pressure-operated switch causes the oil pump to replenish the supply in the tank. This pressure switch automatically shuts off the pump when the pressure in the tank reaches a predetermined maximum.

Fundamentally the action of a governor is as follows: A change in speed causes a change in the position of the rotating flyballs. This change in position moves the distributing or control valve so that oil under pressure is admitted into the regulating cylinder, or cylinders, causing them to move the servomotor piston connected to the gate mechanism of the turbine. This movement of the gates is transmitted back to the distributing valve through the restoring mechanism, bringing the distributor valve back to its neutral, or dead-beat, position after the gates have been moved sufficiently to compensate for the change in load.

Flyballs. The flyballs must respond to the slightest possible change of speed of the unit and thus produce an initial movement of the control valve employed, causing movement of the servomotor that actuates the control mechanism of the turbine. The sensitiveness of this element is of primary importance. Any delay in response produces an initial speed change (rise on load rejections and drop on load increases). This in turn causes delay in the movement of the control mechanism—a lag, which is likely to produce "hunting," in that the movement of the control mechanism, in order to catch up with the delay, becomes excessive. If the control mechanism could follow at such a rate that the resultant output would at all times be equal to the output demand or load, then the mechanism would always be in correct position and no aftermovement would be involved.

To assure highest sensitiveness, the friction of the moving parts producing this initial movement must be the minimum, and the initial movement itself must require minimum effort

Flyballs usually consist of rotating weights supported on a shaft so that the centrifugal force causes them to move outward. This outward movement is resisted by springs and linkage so that the control valve is held in neutral position at the normal speed of the unit. If this speed changes, however, the piston of the control valve changes its position and thus produces movement of the gates by admitting fluid to one side of the servomotor piston Figure 19 shows one of the high-speed flyball heads

A factor contributing materially to the high sensitiveness of flyballs of this design is the fact that the centrifugal forces of the weights are acting *directly* upon the spring thereby eliminating bearing friction

Small flyballs naturally have very little power and would be inadequate to actuate the control valve directly An auxiliary pilot-operated servomotor is therefore disposed between the two, and thus furnishes ample energy at once for prompt movement of the control valve mechanism The rotating stem of the flyballs operates directly as a pilot for the oil-pressure-operated auxiliary servomotor, which produces ample force to actuate the control valve Since this force is produced with pressed oil, it is entirely independent in amount of the initial speed change and therefore contributes immensely to the sensitiveness and resultant energy of the flyballs

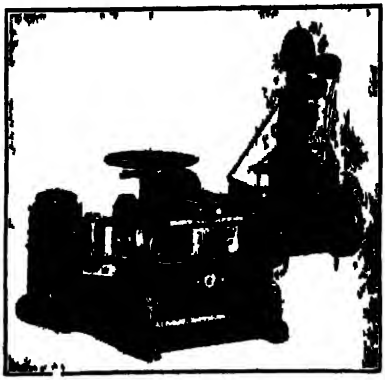


Fig 21 Woodward type H R governor 30 000-ft-lb capacity, equipped with motor-driven flyballs, synchronizing motor and separate hand-gear control with vertical regulating shaft extending through governor base where it connects to horizontal cylinder through a lever and connecting rods.

The flyballs are either directly driven by belts, or are driven by gears from the main turbine shaft with horizontal shaft units, or by gears and jack shaft with vertical shaft units For large units, and particularly for automatic plant operation, a direct motor drive is preferred In Fig 19, the motor is mounted directly on the flyball shaft above the cabinet containing the integral governor parts proper. Figure 21 shows another arrangement of the driving motor on a typical actuator type of governor

A safety device must be provided so that, if the drive to the flyballs fails, the flyballs will not drop and cause the governor to open the gates wide A mechanical device moves the control valve to shut down the unit The motor drive is either a synchronous alternating-current motor or a liberally rated induction motor, so that its slip is negligible; the induction motor has

the advantage of not falling out of step under heavy speed variation, as may happen with the synchronous motor.

Current can be supplied to the motor driving the flyballs from slip rings added to the pilot exciter if it is available. It may be supplied from the generator bus through potential transformers or from an independent, self-excited, alternating-current generator coupled to the main shaft or mounted directly on top of the exciter of a vertical-shaft generator. Speed switches and the magneto for an electric tachometer may also be combined with the pilot exciter. When driven from the generator bus, the governor motor is susceptible to the effects of power surges, and proper protection is to be provided.

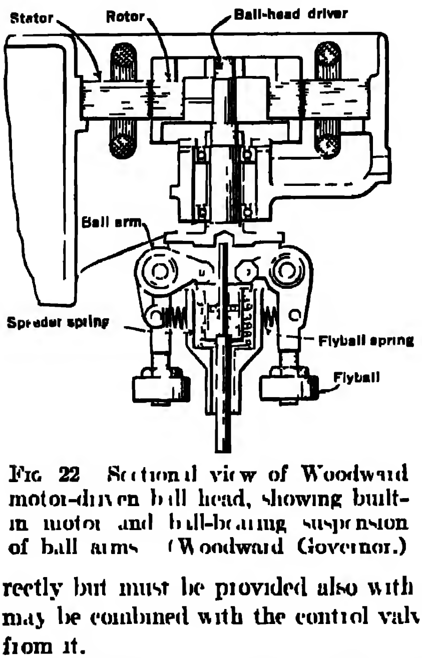


FIG. 22. Sectional view of Woodward motor-driven ball head, showing built-in motor and ball-bearing suspension of ball arms. (Woodward Governor.)

The motor may be built directly into the flyballs as shown in Fig. 22. It is essential, of course, that the arrangement be such that the speed of the flyballs has a definite relationship to the speed of the turbine.

Control Valve. The control valve (see Fig. 20) is a spool-shaped sliding valve that serves to distribute the fluid under pressure to the operating side of the piston of the servomotor. If displacement of the servomotor is very great, the control valve usually becomes so large that it cannot be operated directly but must be provided also with a pilot auxiliary. This pilot auxiliary may be combined with the control valve itself, or it may be entirely separate from it.

Figure 23 illustrates one of the control-valve standards of the Woodward Governor Company.

Servomotor or Regulating Cylinders. The servomotor is a fluid-pressure-operated piston in a cylinder mounted on, or combined with, the body of the governor, as shown in Figs. 21 and 24. The piston rod is connected either to the gate regulating shaft or directly to the shift ring by suitable connecting rods.

With large governors, exceeding 35,000 ft-lb for the Allis-Chalmers governor and 60,000 ft-lb for the Woodward, there are two servomotor cylinders, as shown in Fig. 8; the piston of these two cylinders located on or near the casing is actuated by the control valve located within a cabinet also containing the flyball assembly and controls. The cabinet actuator is usually either on the generator floor above the level of the regulating cylinders or near the turbine. For large units the cabinet contains also such elements

as control valve and flyballs, oil pump, pressure and receiver tanks combined into one group, either for one turbine unit or for two. In such cases the mechanism is mounted on a combination base and sump tank of fabricated steel. On the front of the cabinet are located the controls and indicators, including the standard devices, to be manipulated by the plant operator, and also the necessary instruments for indicating operating pres-

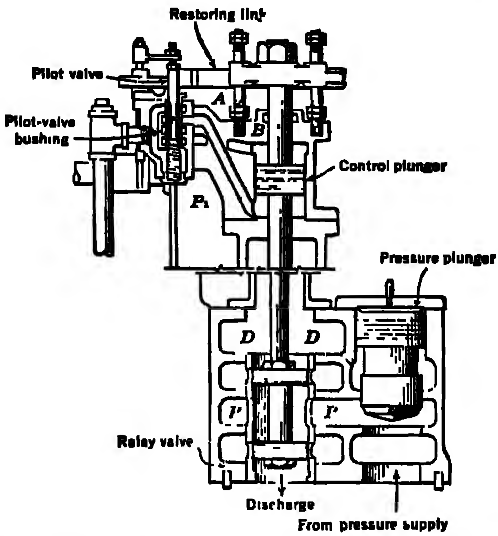


FIG. 23. Sectional view of pilot and relay valves of the Woodward cabinet type of actuator, showing restoring link between the relay-valve plunger and the pilot-valve bushing. Movement of small pilot-valve plunger by flyballs causes control plunger to move relay valve a much larger amount, which admits oil to servomotor to move gates. With this arrangement, speed changes of 1/100 of 1% or less will operate relay valve and cause movement of gates. (Woodward Governor.)

sures of the turbine proper and of the oil-pressure system. Figure 26 shows this type of cabinet of Woodward and Allis-Chalmers governors.

Restoring Mechanism. The restoring mechanism connects the regulating piston actuating the flow-controlling means with the control valve. When the regulating cylinder is combined with the governor body, as in Fig. 24, this mechanism, as well as the flyballs and control valve, are combined into one unit. When two regulating cylinders are mounted directly upon the turbine casing or upon a foundation near by, the other elements are mounted upon a so-called governor actuator, which is usually located on the generator floor or at some distance from the turbine.

To hold the turbine at no-load gate opening or at full load, a certain difference in speed of the unit is required. This difference is the "droop," or the relative speed variation between no-load and full-load operation.

Without restoring mechanism this droop would be zero and would cause the governor to hunt.

The speed droop can be altered, within limits, by adjustment of the compensating relay, which consists of the spring and dashpot, shown at the left of Fig. 20. Thus, by the proper regulation of the spring tension and dash-

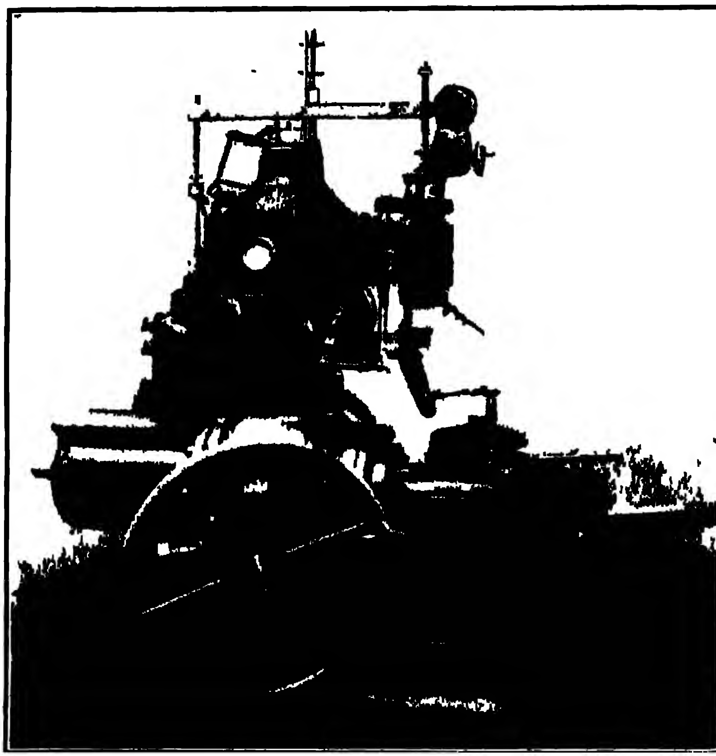


FIG. 24. 18,000-ft-lb capacity oil-pressure governor for vertical turbine. Regulating shaft extends up through base with operating lever keyed to it. Separate hand control through bevel gears on worm.

pot by-pass, the movement of fulcrum *A* can be controlled so that the restoring mechanism more quickly returns the control valve to the neutral position. The compensating relays can be adjusted for droops from 6% to 1% or less.

The design of the governor and the mechanical details of the relay mechanism vary greatly and will be discussed in a later paragraph.

Fluid Pressure and Pump Arrangement. The standard fluid-pressure medium for actuating the governor and control mechanism is oil. Where clean water is available at sufficient pressure, it is sometimes used with a small amount of soluble oil or potassium bichromate in solution. With water, the

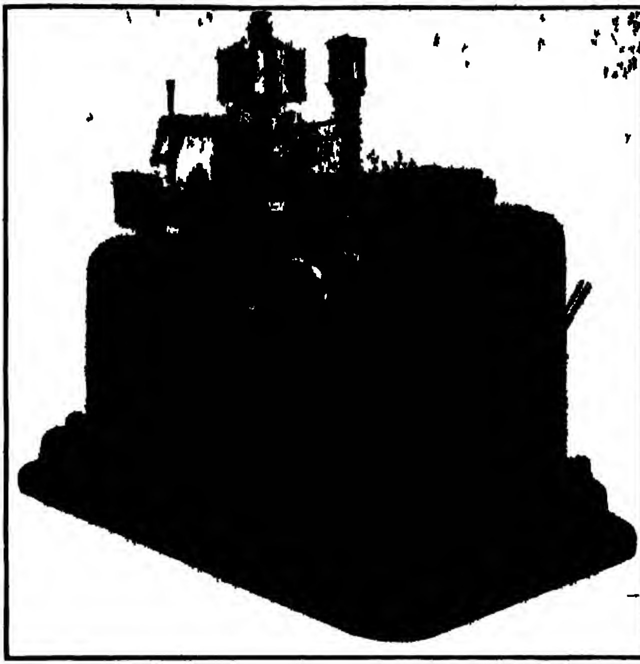


FIG. 25 Control stand for 100,000-ft-lb capacity governor, showing gate indicator, tachometer, and hand- and electric-operated controls (Pelton.)

pump pressure and receiver tanks can be omitted. Water is not a good lubricant. Also, it tends to cut the ports of the control valve so that special precaution must be taken in the selection of materials and in provision for easy renewal of parts subject to wear. Water serving as the pressure medium impairs the sensitiveness of the governor. A more undesirable feature, however, is the danger of "sticking," which may cause runaway of the unit. Many installations using water have eventually changed over to oil.

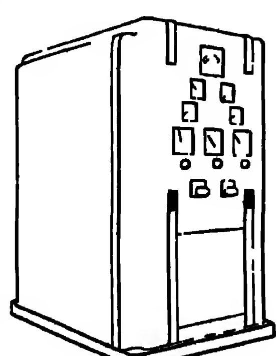


FIG. 26 Cabinet type of governor stand.

One of the main requirements for good performance is that the oil does not "break down" as decomposition produces tar deposits in the control valve and thus causes sticking. Acids that corrode the material also result in sticking of the governor. Oil with as nearly constant viscosity as possible is advisable, since thin oil under high temperature requires entirely different adjustment for maintenance of a desired rate of movement of the governor from that required for heavy oil. This principle applies especially to dash-pot oil,

which controls the rate of movement. Too thin an oil results in serious hunting. Only oil receiving the indorsement of the governor manufacturer should be used.

The oil-pressure supply consists of a pump, pressure and receiver tanks, piping, and accessories. These may be arranged as follows:

1. An individual pump set for each unit, especially where only a few units are located in a plant, or where the unit is of large capacity.
2. A duplex arrangement for supplying oil pressure to two units.
3. A central oil-pressure system supplying oil to all units of the plant.

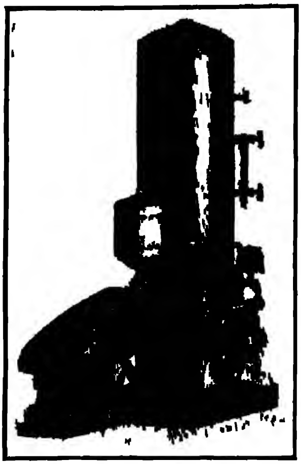


FIG. 27. Motor-driven oil-pressure pumping unit for 30,000-ft-lb governor, showing location of pressure and receiving tanks, gear-type oil pump, relief valve, gage glass, and pressure gage

For isolated units a pumping set such as that shown in Fig. 27 can be belt-driven from the horizontal main shaft or from a jack shaft, or motor-driven, as shown, either by direct coupling, by a silent chain, or by tex-rope drive. The pressure and receiver tanks are mounted on a common base, and the oil pump is attached to the base forming the receiver tank.

Figure 28 shows a motor and impulse-wheel-driven pumping unit that can be operated in the event of electric-power failure The installation of such a unit insures an emergency supply of pressure oil.

For large-capacity individual pump sets the pressure and receiver tanks are separated and the pump is usually motor driven For high-head plants the pump drive may be a small impulse wheel directly coupled to the pump

The duplex arrangement has come into much favor as it permits of an added flexibility, in that one pump may serve both units in an emergency, or each pump may be of sufficient capacity so that the other may serve as a stand-by only.

The central oil-pressure system proves economical especially where the units are not called upon to handle heavy momentary load fluctuation In such a case two pumps will prove sufficient if each is of a capacity to take care of normal operation of all units in the plant An unloading valve is provided for each pump, permitting the pump to float idly, that is, under no appreciable oil pressure, until the pressure in the central pressure tank drops below a desirable minimum, or it may be combined with a pressure switch that causes the pump and motor to stop completely.

The oil pumps proper are now almost exclusively of the gear or screw type, plunger pumps having been entirely eliminated.

The standard maximum oil pressure was formerly 150 to 200 lb per sq in., but in modern governors of larger sizes 300 lb has become normal.

In the event of complete absence of oil pressure, a mechanical hand control device combined with the servomotor of the governor may serve the purpose for small units. For larger units and especially in connection with actuators, hand oil pumps have been provided for emergency use when no power is available until the unit is started up.

Pressure and receiver tanks should be designed to prevent air from becoming mixed with the oil supply, as otherwise a compressible medium results which impairs the correct action of the governor.

The so-called open system is now exclusively used, in which the return oil discharges into the receiver under atmospheric pressure. The departure from the so-called closed system under vacuum has greatly eliminated the breaking down of the oil with subsequent failure of governor to perform.

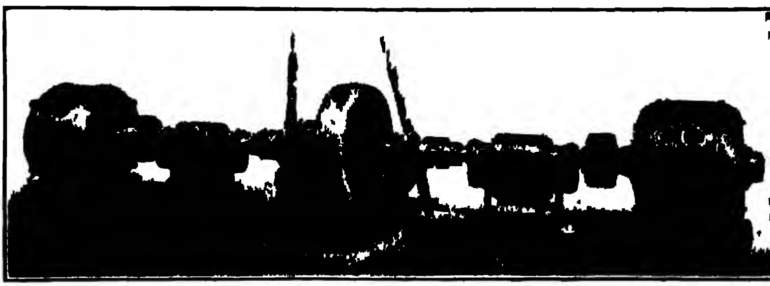


FIG 28. Motor and impulse-wheel-driven rotary pumping set for large central-governor oil-pressure system with clutch couplings on both sides of impulse wheel, so that it can be used for driving either pump in the event of failure of electric power supply.

The piping should be given special care. It should be cleaned of any scale before using as scale causes considerable damage when carried to the control valve, etc. Threaded pipe fittings should be avoided. Flanged or welded joints are suitable. All bends should have liberal radii of curvature to reduce losses in pressure. Steel drawn tubing is customary in present-day practice.

12. Governor Capacity. The required foot-pound capacity of the governor depends on the type and size of turbine that it is to control and also on the nature of the load to which the turbine is subjected. The rated capacity of the governor is measured in foot-pounds.

$$C = \frac{\pi(D^2 - d^2)psn}{4} = 0.785(D^2 - d^2)psn \qquad [4]$$

in which C = capacity of the governor, in foot-pounds;

D = the diameter, in inches, of the cylinder bore;

d = the diameter, in inches, of the piston rod projecting through the packing box of the cylinder;

p = oil pressure, in pounds per square inch, measured at the pressure inlet to the control valve;

s = the stroke, in feet;

n = the number of cylinders, usually 2 or 1.

The approximate required governor capacity C_1 can be computed from the formula

$$C_1 = \frac{K \times P}{(H)^{1/2}} = \frac{K \times P \times H}{(H)^{3/2}} = K \times P_0 \times H \qquad [5]$$

in which C_1 = required capacity of governor, in foot-pounds;
C = maximum required capacity of governor, in foot-pounds, for maximum head;
K = a coefficient;
P = capacity of the turbine in horsepower;
P_0 = capacity of the turbine at 1-ft head = $P/H^{3/2}$.
H = head, in feet;
K = 40 for outside type of gate-operating mechanism for Francis turbines;
K = 50 or more for inside or open-flume type of gate mechanism for Francis turbines;
K = 55 for propeller turbines;
K = 30 to 35 for stream deflectors of impulse wheels;
K = 45 to 60 for needle control of impulse wheels, depending greatly on design. (European makers, in order to cut down governor capacity to a minimum, apply springs to counteract the variable needle forces.)

This rating fixes the characteristics of a governor only as concerns the force developed along the stroke (foot-pounds), but it does not contain the elements of time. One governor may require 6 seconds or more to close the wicket gates of a turbine, whereas another may be capable of accomplishing it in 2 sec. In plants with long penstocks, the governor must move slowly in order that serious water hammer does not occur (see Chapter 34). A more precise method of rating the governor than that given in Eq. 5 is to include the time of closure, thus obtaining the work per second done by the servomotor piston, or

$$C_2 = \frac{C_1}{T} \qquad [6]$$

in which C_2 = required capacity of the governor, in foot-pounds per second (energy);
C_1 = foot-pounds capacity of the governor, as given by Eq. 5;
T = time in seconds desired for the full stroke of the piston of the servomotor. This is generally called "the total governor time" and is the time required for the governor to shut the turbine gates from full-open position after full load has been suddenly thrown off. (See Section 13.)

13. Speed Regulation. In determining the size and characteristics of a governor for a hydroelectric unit, it is essential that the required governor

closure time and the permissible speed rise when full load is dropped be specified, as well as the Wr^2 of the unit and its power, head, and speed. Allowable pressure rise in the penstock (generally about 20%) determines minimum closure time, which may be as low as 1 sec for open-flume settings of small turbines or as great as 15 sec for long penstocks without surge tanks for part-load change. In addition to the sensitiveness of the governor, the regulating results will depend on the Wr^2 of the entire connected load and the type and size of load changes.

It is usually essential that the speed fluctuations be small for momentary changes in load, as most types of consumers' load require steady speed. It is the function of the governor (Section 11) to hold the speed of the unit as close to normal as practicable under changing load conditions. Speed increase on sudden dropping of the full load should be limited to a maximum of 30 to 36% of normal speed. Many factors are involved in this problem. The primary factors are

Wr^2 = the weight of the rotating parts multiplied by the square of the radius of gyration. This is sometimes referred to as the flywheel effect, or moment of inertia of the revolving mass.
P = the rated horsepower capacity of the unit.
xP = the change in load, in horsepower.
x = the ratio of the change in load to the rated power capacity, $(P' \pm P)/P = \pm x$.
t_e = the time during which the load change occurs, in seconds.
t_0 = the delay between the initial speed change and the start of the action of the governor piston, in seconds.
t_x = the time required for the governor piston to move to adjust the turbine gates to the new load, in seconds.
T = the total governor time, in seconds, required for the governor to shut down the turbine gates from the full-open position after full load has been suddenly thrown off. $T = t_0 + t_g$.
t_g = time for governor piston to move from full-open to closed position.
N = normal speed of the unit, in revolutions per minute.
N' = speed of unit after speed change occurs, in revolutions per minute.
N_r = runaway speed.
i_s = speed-change ratio or relative speed change
= $(N' - N)/N$ for load decrease
= $(N - N')/N$ for load increase.
C = flywheel constant for the unit = $(Wr^2N^2)/P$.

Secondary factors will be set forth in Section 14.

Governors are designed and adjusted so that the total time of closure T is from 1 to 15 sec or more, the upper value of time of closure being applicable to high heads and long penstocks. Thus a 3-sec governor would be one that, when the load was dropped, would close in 3 sec. In general, speed regulation is closer if governor time is as short as practicable, but it is essential only in special cases that it should be less than 3 sec. In any given case the costs of governor with accessories will be greater for a small required value of T than it would if the value of T could advisedly be greater.

When the unit is part of a system connected to a motor load equivalent to many such units, one rotating unit cannot change its speed appreciably without affecting all the other connected generators and motors, unless the unit is disconnected from the system. The influence of a single unit on the whole system, however, is indicated by its performance as an isolated unit that suddenly loses its full load.

The diagram in Fig. 29 shows the difference between the output of the generator and input of the turbine when the load suddenly changes from full to zero in the time t_e. Line ABD indicates the load of the unit during the time of gate closure, T, and line ACD shows the input of the turbine during

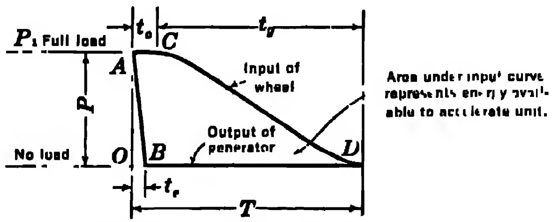


FIG. 29 Diagram showing energy available for accelerating unit when load is dropped in time t_e.

the same time The area between these lines represents the excess energy that accelerates the speed.

The governor requires t_0 sec to respond to the load change, and so the gates do not start to close until point C (time t_0). In many cases this time lag (t_0) may be safely assumed as ¼ sec. It is assumed for simplicity that the gates then close at a uniform rate and that the power output at any instant is proportional to the gate opening, so that CD is a straight line. If the load change is taken as instantaneous, t_e becomes zero. Then, from Fig. 29, the area between the input and load lines is approximately

$$Pt_0 + \frac{P(T - t_0)}{2} = \frac{P(T + t_0)}{2} \qquad [7]$$

Converting this to the customary units of foot-pounds by multiplying by 550, the energy available to accelerate the speed of the unit when full load is instantaneously removed becomes:

$$E = \frac{550P(T + t_0)}{2} \text{ foot-pounds} \qquad [8]$$

If this energy is equated to the kinetic energy in speeding up the unit from N to N' in the time T, an equation for the ratio of the increase in speed to the normal speed, called momentary relative speed rise (i), is obtained as follows for sudden load decrease.

$$i = \frac{N' - N}{N} = -1 + \sqrt{1 + \frac{1{,}620{,}000P(T + t_0)}{N^2Wr^2}}$$

or

$$i = -1 + \sqrt{1 + \frac{1{,}620{,}000(T + t_0)}{C}} \qquad [9]$$

For sudden load increase the relative speed drop is

$$i = \frac{N - N'}{N} = 1 - \frac{N'}{N} = 1 - \sqrt{\frac{1 - 1{,}620{,}000(T + t_0)}{C}} \qquad [10]$$

The value of the flywheel constant, C, as already defined, is an indication of the stability of regulation of the unit. C is derived from the equation

$$C = \frac{N^2Wr^2}{P}. \qquad [11]$$

Past experience has shown that, if C is less than 3,000,000, the governor is likely to hunt even under nearly constant load. If the penstock is very short and there is little danger of surge or water hammer so that the governor time, T, can be made very short, then values of C of 4,500,000 or more give satisfactory governor regulation. For propeller turbines the value of C should be at least 5,000,000 for stability at constant head and load.

For partial load changes the corresponding speed variations are less. Assuming that the governor delay and closure time both vary in proportion to the load change, then the total governor operating time may be assumed to be proportional to the load change. If x represents the ratio between the partial load change and full load change, then Eqs. 9 and 10 for the relative speed rise and drop, respectively, with a partial change in load, xP, become

$$i_x = -1 + \sqrt{1 + \frac{1{,}620{,}000}{C} \times [t_0 + x(T - t_0)]} \qquad [12]$$

for relative speed rises on sudden load rejections, and

$$i_x = 1 - \sqrt{1 - \frac{1{,}620{,}000}{C} \times [t_0 + x(T - t_0)]} \qquad [13]$$

for relative speed drops on sudden load increases.

Note: The delay t_0 does not diminish at partial load changes, but only the actual governor time, $T - t_0$.

A simpler equation for the speed-change ratio that is commonly used is as follows:

$$i_x = \frac{810{,}000x^2T}{C} \qquad [14]$$

applying to speed rise as well as speed drop. This equation is obtained from Eq. 12, assuming, however, incorrectly, that the average speed during the load change equals the normal speed and that the governor delay, t_0, is zero.

Equation 14 should not be used when it gives high relative speed changes, such as are caused when C is small and T is long, as results would be seriously in error.

Equations 9, 10, 11, 12, 13, and 14 are adequate only for moderate relative speed changes, and when the head H remains constant during the process of regulation. These equations are applicable only to sudden load rejections from full or part load, to zero, and for sudden load increases from zero to full, or part load to full, and are not correct for intermediate part-load changes. The reasons for above limitations are set forth in Section 14.

Gradual Pressure Changes in Penstock. It has been assumed that the pressure changes take place within the period of initial action of the governor. Any pressure changes occurring much more slowly than the governor movement will not seriously affect the calculated results but will have an aftereffect dependent on the amount and rate of pressure change.

Such relatively slow pressure changes are caused by wave accumulation in open flumes where there is a free water surface, or in pipelines in which there are surge tanks. In such cases, if the maximum pressure rise or drop produced at the turbine occurs long after the initial movement of the governor, it must be investigated in accordance with the principles discussed in Chapters 34 and 35.

Power Surges. With long transmission lines, electric disturbances due to sudden load changes may also react upon the governor of the prime mover and may accumulate to such extent that the generator falls out of step. It is essential that in the selection of the equipment these factors be considered.

14. Summary of Speed Regulation Problems. Several secondary factors affect speed regulation. The magnitude of their effect may be considerable, especially in a system that does not consist of a multiplicity of generating units with a diversified load. These secondary factors are

1. Rate of change of output by action of governor.
2. Runaway speed.
3. Head variation.
4. Windage and friction of revolving parts.
5. Change of head due to change of flow.

These secondary factors are discussed and formulas for determining their magnitude in various situations are given in Ref. 8.

It is realized that the problem of speed control, especially of turbines receiving water through a pressure conduit, is a very intricate one, requiring step-by-step, lengthy computations for close end results. These calculations should be done by an expert. (For explicit step-by-step computations see the excellent paper by E. B. Strowger and S. L. Kerr, entitled "Speed Changes of Hydraulic Turbines for Sudden Change of Load,"* and also some detail examples in Ref. 8.)

* Spring meeting of American Society of Mechanical Engineers, San Francisco, Calif., 1926.

If approximate calculations are sufficient, the results of simplified equations assuming a safe margin are plotted in two curve charts, *A* and *B* in Fig. 30, for sudden load rejections.

Chart A shows as ordinates the ratios LV/H, where L is length of pressure pipe in feet, V the full-load average velocity of water in feet per second over the total length L, and H the net head in feet at full-load flow of water. The relative pressure rise in percentage above the net head H is plotted as abscissas.

For each individual pressure rise or abscissa of chart, the value $t_g H/LV$ or $t_g/(LV/H)$ remains constant, and so the value LV/H is directly a multiple of the governor time T. Thus the value LV/H or the ordinates can be taken from the curves of the governor time T of 1 to 15 sec or for fixed values LV/H the resultant per cent pressure rises for a selected governor time can be read.

Chart B shows the maximum percentage of relative speed rise for ordinate values C/T at various percentages of maximum pressure rise as abscissas.

C being the flywheel constant N^2Wr^2/P (Eq. 11), from which the flywheel effect of rotating masses of the unit can be computed for the respective speed N of the unit and normal output P, or for a fixed ratio C/T, the per cent pressure rise can be determined for a fixed per cent relative speed rise

The result of Chart B is computed from the simplified formula

$$\frac{N' - N}{N} = -1 + \sqrt{1 + \left(\frac{1{,}620{,}000}{C/T}\right)\left(1 + \frac{\Delta H}{H}\right)^{3/2}}$$

Assuming $C/T = 3{,}500{,}000$, the pressure rise 30%, and the governor closure time 3 sec, we have for the speed rise

$$\frac{N' - N}{N} = -1 + \sqrt{1 + \left(\frac{1{,}620{,}000}{3{,}500{,}000}\right)(1.3)^{3/2}} = 0.30$$

In this example, the known quantities for use in Fig. 30 are the ratio LV/H and C/T, and the pressure rise in the penstock of 30% (on dropping of the load), which we do not wish to exceed for reasons of safety.

Taking the value of LV/H in Chart A, Fig. 30, as 25.5, we proceed horizontally, as indicated by the dotted line, until we intersect the permissible pressure rise, in this case 30%. Here we read that the required total governor time is 3 sec. From this point we move vertically along the 30% pressure-rise line in Chart B, as shown by the dotted line, until we reach the given value of the ratio C/T, which in this case is 3,500,000, as shown by the dotted horizontal line. The intersection of the vertical and horizontal lines gives the approximate speed rise that will occur on the dropping of the entire load—in this case about 31%, which is a fairly conservative value.

Figure 30 shows the hydraulic designer promptly the approximate effect on pressure and speed regulation of any change that he may consider in the

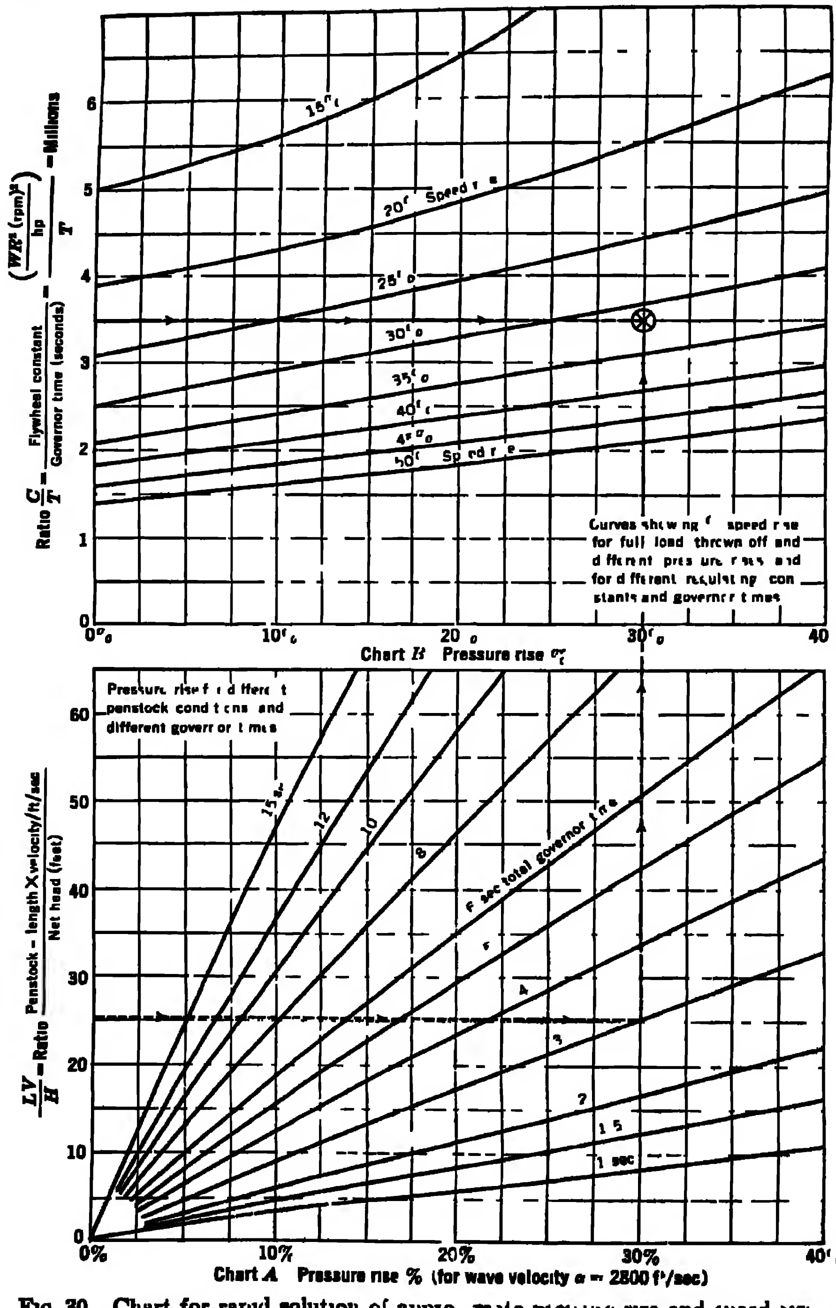


FIG 30 Chart for rapid solution of approximate pressure rise and speed rise

several variables concerned: diameter of penstock and velocity, governor closing time, Wr^2 of hydro generating unit, and revolutions per minute of unit.

15. Plants without Governors. There are a number of small plants in operation that are not equipped with governors, where the only control of the unit is by hand mechanism or by a motor. One of these plants was equipped in this manner because of the extreme penstock conditions and the desire to save the additional cost of the surge chamber or pressure regulators. Since these plants were to operate in parallel with other units provided with governors, they could be set for a constant output; and their load was varied only to maintain a margin for regulating on the governor-equipped plants or to suit the flow.

The most essential feature in a unit of this kind is that all of the rotating parts be designed to stand prolonged runaway under the maximum head conditions possible with the gates wide open. Although this condition may occur only at rare intervals, such as when there is line trouble or when for some other reason the entire load is lost, it is sure to exist at some time and the units should be designed accordingly. Some of these plants, which have motor-operated mechanisms, are equipped with an overspeed flyball control switch that starts the motor in the closing direction when the speed reaches approximately 10% or some other set value above normal. The gate mechanism is designed to close in about 50 sec.

The scheme is particularly applicable to small plants that form a part of a big power system containing a large number of plants, many of whose units are governor-controlled. For a small unit the saving in capital cost by the omission of the governor may be relatively great, but with large units such saving in cost would be immaterial.

16. Automatic Control Systems. Nowadays, on account of the high cost of labor, there is a tendency toward automatic equipment. Such equipment greatly reduces the number of operators required and makes it possible to develop small hydroelectric sites that could not warrant their existence if the salaries of a crew of operators were to be maintained. There are three general types of automatic control.

Full Automatic Control. A unit designed for full automatic control is usually located a considerable distance from the point of power consumption, and the method of operation is approximately as follows: As long as the transmission line to which this unit is connected is energized, the unit will operate. When the switches connecting the transmission line to the system are thrown in, the first operation generally is to start the governor pump, which builds up oil pressure and slowly opens the gates of the turbine. As the unit comes up to speed, flyball-controlled switches connect the generator to the line at approximately synchronous speed through amortisseur or damper windings that limit the flow of current through the generator until it has been pulled into phase. The gates of the turbine then continue to open up until that point in the gate opening has been reached which corresponds to the speed of the system and the load setting of the governor. The unit then con-

tinues to operate until it is shut down by disconnecting the line switches at the other end of the line or by some of the protective devices on the unit itself.

Protective devices may include some or all of the following: bearing temperature relays, generator-winding temperature relays, water-pressure relays, overload relays, reverse-current relays, low-voltage relays, ground relays, and relays connected to any part of a hydroelectric unit that it is desired to protect. If the unit has been shut down by some protective device, it cannot again be started up until a repair man or operator has corrected the trouble. Units so equipped are usually inspected once each day, at which time they are greased and the lubricating systems are gone over.

Remote-control Stations. Remote-control stations are chiefly the outgrowth of large units that are controlled from the switchboard rather than from the operating floor, as the operator at the switchboard is in a better position to watch the operation of the unit, by observing the action of the various load meters, than is the turbine operator on the turbine floor. Operation from the switchboard usually consists of controls to the synchronizing and load-limiting devices on the governor, the switch for the governor oil pump, the controls for the electrically operated switches connecting the generator to the transmission line, as well as contact switches for shutting the unit down quickly in an emergency. These same electrical connections may be run to a distant station, where the operator, observing the action of the unit through the frequency meters and load meters, can control its operation These various controls are generally handled through separate wires for each, although a single pair of wires and a selector mechanism, such as are used in automatic telephone work, can be employed. Frequently this remote-control system is combined with an automatic-control system, the unit being started up by remote control and the load adjusted to suit demands or according to water supply, and the automatic equipment being provided to take care of any emergency shutdown.

When a unit is operated from a penstock so long or subject to such high water velocities that only a very gradual change is permissible to hold the pressure variations within safe limits, the rate of change of discharge either by hand or by emergency shutdown means must be very gradual. A slow closure, however, may cause a complete runaway of the unit, and although this may not harm the unit itself, it yet may damage the pipeline, because the characteristic of the turbine may be such that on a runaway its discharge is materially reduced, and, if the inherent flywheel effect of the detached unit is small, it may attain the runaway condition so quickly that the meanwhile reduced discharge causes a serious water hammer. In this event a pressure regulator is indispensable.

Float Control. The third type of automatic equipment is designed to operate the unit so as to utilize most effectively the full flow of the stream. Two or more float-operated contact switches are provided, either in the forebay or connected to the penstock, and actuated by pressure. When the water in the forebay rises to a predetermined height, the unit is started up, synchro-

nized, and placed on the line, carrying its full rated load until the water in the pond reaches a predetermined low level, at which time the unit is shut down tight.

Combinations of this system may be arranged by adjustment of the full load that the unit may carry to suit approximately the average flow for the existing season; or five or ten different level-operated switches may be provided, so that when the water is at the maximum height the unit carries a full load, when it drops 1 ft to the next switch the unit carries 0.9 load, and

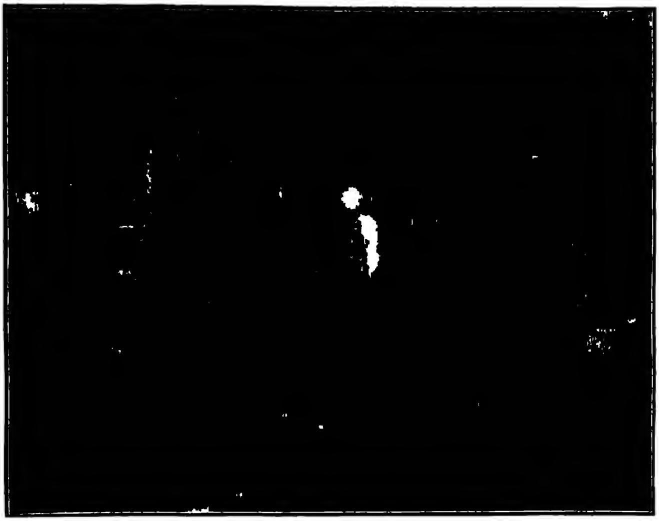


FIG. 31. Float-controlled unit of Lihue Plantation Co., Lihue, Hawaii. 1070-hp, 208-ft head, 900 rpm. (Allis-Chalmers.)

when it drops 10 ft the unit is shut down entirely. With this last system the unit would operate practically continuously and would carry a load corresponding approximately to the stream flow. This system includes the protective devices similar to those of the remote-controlled and automatic-controlled stations. It is used also in connection with both the automatic and remote-controlled systems to regulate the amount of load that the unit may take.

Float control is necessary also when no forebay storage is available, and in this event the unit at all times must be adjusted according to available flow irrespective of maximum load demand. The float acts upon the load-limit device of the governor reducing gate opening when flow decreases. When all the available flow must be delivered continuously to the tailrace of the unit for irrigation purposes, a water-wasting by-pass must be provided and so interconnected that first the action of the float reduces wastage of water through the by-pass until it is stopped completely, and then the float acts

upon the load limit of the governor until it has reduced the flow through the turbine to equal that available, and vice versa. Figure 31 shows such a unit installed at the plant of the Lihue Plantation Company, Lihue, Hawaii, rated 1070 hp under a 208-ft head at 900 rpm (Allis-Chalmers).

17. Pressure Regulators. Pressure regulators are devices used with hydraulic turbines where long penstocks are involved; they are to prevent rapid velocity changes in the pipeline in that they open when the turbine gates close. When surge tanks, which materially reduce the pressure section of pipe, are used, pressure regulators are generally unnecessary.

Pressure regulators may be designed for water-saving action, so that they close slowly after steady conditions have been attained, or for water-wasting action, so that they maintain approximately constant velocity in the pipeline and close only when the turbine gates are again opened. They are not responsive to pressure variations in the pipeline but are operated solely in connection with the control mechanism of the turbine (either by hand, electric motor, or automatic governor).

A pressure regulator consists of the main outlet and of its operating device and connections to the turbine. Depending on design, physical size, and operating head, the main outlet may be actuated directly or by a servomotor. Three principal types of main outlets are in use:

1. The balanced-cylinder-gate type.
2. The mushroom-disk type.
3. The needle-valve type.

The balanced-cylinder gate is suitable only for moderate physical size and for low and moderate heads. The mushroom-disk and the needle-valve types are suitable for any size and head.

The water-wasting pressure regulator opens and closes as the turbine gates close or open, irrespective of rate of movement; they remain wide open when the turbine gates are completely closed and closed when the turbine gates are wide open. If the discharge capacity of the pressure regulator equals 100% of that of the turbine, and if its discharge curve is inversed to that of the turbine at all intermediate gate positions, then the total discharge through the turbine and pressure regulator remains equal to the full discharge of the turbine so that the velocity in the penstock remains constant. If the total discharge capacity of the pressure regulator is only 80% of the total discharge capacity of the turbine, a reduction of 20% in the discharge through the penstock takes place on full-load rejection, and a 20% increase in discharge through the penstock occurs on full-load increase.

Likewise, any discrepancy in synchronism during the control movement results in a corresponding change in velocity of water in the penstock and may produce disturbing pressure variations if the penstock conditions are severe. The water-wasting pressure regulator must be used when the flow is to be maintained (for irrigation, etc.) irrespective of load conditions or

where the load fluctuates so rapidly that the water could not be accelerated on sudden load demands without causing serious pressure decreases in the penstock.

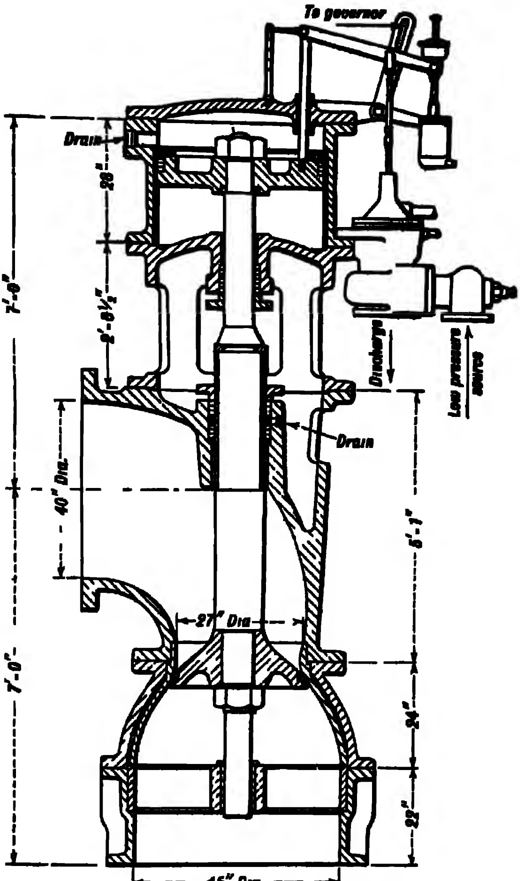


FIG. 32 Governor-controlled pressure regulator for water-saving or water-wasting operation. Pilot valve at right is operated from gate mechanism and when gates close releases pressure from within cylinder, allowing mushroom valve to open to maintain constant velocity in penstock. Dashpot then controls pressure regulator closing time or, if locked, pressure regulator remains open, wasting water. (Allis-Chalmers)

The water-saving pressure regulator is designed to open only when the turbine gates are closed at such speed as would cause excessive pressure rises in the penstock and then to close so gradually that no undesirable pressure rise will be set up in the penstock. This is accomplished by an oil dashpot between the turbine control mechanism and the main outlet, which on slow

movement of control absorbs the movement so that the outlet remains closed, or on fast control movement transmits it with practically no delay and then slowly closes the outlet. Locking of oil-dashpot piston with cylinder eliminates the relative movement so that the outlet moves synchronously (water wasting) with the control of the turbine.

Interlocked control is necessary when the pipeline conditions are so serious that no appreciable delay between the action of outlet and control of turbine

FIG. 33. Hoover Dam pressure regulator for unit A7. (Allis-Chalmers.)

is tolerable. This design prevents the turbine control from closing if the main outlet sticks and refuses to open, and thus it retards the closing so that dangerous pressure rises are averted. A powerful oil dashpot and connections are required in order to counteract all forces produced by the governor.

The noninterlocked mushroom type, shown in Fig. 32, consists of a mushroom-shaped disk mounted on a stem and held against the outlet of an elbow connected usually to the turbine casing. The stem projects through the elbow into a cylinder either directly combined with the elbow or separate in case other than penstock pressure is used. The stem carries a piston holding the outlet closed when the space below the piston is held under full penstock pressure, but the outlet opens when the pressure is reduced by a control valve operated by a relay mechanism between turbine control, oil dashpot (unless directly water-wasting), and main outlet.

The needle-valve type (see Section 9, Chapter 26) differs from the mushroom type only in that it opens upwardly whereas the mushroom type opens downwardly, the inlet elbow being designed accordingly. Some designs have the control valve built directly with the pressure regulator, with the disadvantage that the hydraulic control becomes inaccessible for inspection or repair.

The interlocked type in both mushroom and disk form, as well as in needle-valve design, has successfully served in connection with the Hoover units. The interlocked mushroom type as developed by Allis-Chalmers is shown in Fig. 33. This pressure regulator is designed to open fully in 4 sec's governor time, requiring only moderate force over the control by reason of its hydraulic action. The oil dashpot located on top of the pressure regulator with piston adjustably connected to the oil servomotor actuating the turbine gates is provided with an adjustable by-pass jet so that the main outlet of the pressure regulator is closed automatically in 50 sec from wide-open position. However, in the event of failure of the main outlet to open, the dashpot becomes interlocked with the outlet so that a high oil pressure is at once produced under the dashpot piston, resisting the force of the governor. At the same time this higher oil pressure causes the oil by-pass to close tightly. Thus, the governor is retarded to a rate of close to 50 sec so that no appreciable pressure rises take place in the penstock during the closure of the turbine gates.

18. Absorption of Kinetic Energy from Pressure-regulator Discharge. The kinetic energy contained in the water discharging from the pressure regulator may be so considerable that the discharge cannot be allowed to occur freely into the tailrace. For instance, 80% discharge capacity of the Hoover Dam pressure regulators represents an output equivalent to 80% of 115,000 or 92,000 hp, which, if released into the tailrace, would cause destructive disturbances. It is necessary to transform this energy by reducing the velocity of the discharging water. This is accomplished with so-called energy absorbers of various designs, used as early as 1913 in connection with the pressure regulators of the double overhung impulse units at Big Creek plants Nos. 1 and 2. (See Chapter 38, Table 1.)

The kinetic energy still contained in the water discharging from the pressure regulator proper must be reduced before the water reaches the tailrace to avoid destructive effects. For high heads this reduction has been successfully accomplished with energy absorbers. The water discharges into a T-shaped pot with an omega-shaped bowl at its bottom which turns the jet stream lines practically 180 degrees around, against the arriving water. Thus, the pot becomes filled completely and causes the water to leave through the horizontal outlet of the T. Other means in the form of a grille made of channel irons or railroad rails placed into the outlet of the discharge pot have also given entirely satisfactory results.

The stationary cone of the Howell-Bunger valve is quite effective in destroying the kinetic energy of the jet. These valves (Fig. 24, Section 9, Chapter 26) sometimes serve as pressure relief valves.

19. Valves. Valves of the butterfly or needle (Johnson) type are used at the inlet of the spiral casing of many medium- and high-head units. They form an integral part of the hydraulic turbine equipment and are usually purchased with it. The safe and reliable functioning of these valves is an important item in the operation of a hydraulic plant, and their design and construction should be given as great care as other parts of the hydraulic equipment. The various types of valves and the details of their construction and methods of operation are discussed in Chapter 26 of this book.

20. Spare Parts. Some spare parts are desirable for every hydraulic turbine installation. In a plant containing a number of identical units, one spare runner should be kept on hand. For Pelton wheels there should be spare buckets. From two to six spare guide vanes are advisable and at least one complete set of guide-vane links or shear pins if they are of the breaking type. With water-lubricated bearings, it is desirable to carry an additional bearing shoe already lined with lignum vitae or other bearing material, and bored to size. With lignum vitae, the shoe should be kept in moist sawdust to prevent drying out. It is usually not considered necessary to carry a spare babbitted bearing as this type causes very little trouble, although with the ring-oiled horizontal bearing an extra bearing shell is sometimes carried in stock. For the governor equipment on large units, a spare set of pump gears and a regulating valve are desirable, but on small units scarcely any spare parts need be carried for the governor as the unit can be run on the hand control in the event of governor trouble.

The above applies to readily accessible plants. The whole matter of spare parts is dependent to a considerable extent on the degree of accessibility. Thus, a plant in a remote part of South America should naturally carry more spare parts than one in Pennsylvania.

21. Miscellaneous Auxiliary Equipment. Other items of auxiliary hydraulic equipment sometimes required in a powerhouse include head gates, penstocks, drain valves and piping, ejectors, air compressors, wrenches and tools, inspection manholes, pumps for pumping out submerged parts below tailwater level, and an adequate means of supplying continuously, winter and summer, such water as is required for cooling governor oil and oil-lubricated bearings, and for lubricating lignum vitae, water-lubricated steady bearings.

Head gates and penstocks are discussed in Chapters 25 and 28.

Drain valves of some sort are required in all water-power plants. Usually simple flap traps are sufficient for open-flume and low-head concrete spiral-cased units. Plate-steel- and cast-metal-cased units, as well as many concrete spiral-cased units, require drains equipped with gate valves, frequently with the operating mechanism led up to the operating floor. Such valves must be of liberal diameter to carry away the maximum head-gate or valve leakage, and must be of rugged design so that they can be opened with full head on the valve and closed again. They should also have a sturdy operating mechanism that will not twist off if some obstruction clogs the valve.

Ejectors are used in some low-head, open-flume, concrete spiral-cased settings known as "siphon settings" to eject air from that part of the inclosed chamber which is above headwater. They must be properly screened to prevent clogging, and the suction pipe to the top of the chamber, as well as the ejector itself, should be protected from damage by "dead heads."

Air compressors are a desirable accessory to provide air pressure for the generator brakes for stopping the unit as well as for replacing the air cushion on governor pressure tanks. In small governors not located at high altitudes, a small amount of air can be bled through the oil pump, but this is too slow a process when it is desired to get a unit into service quickly, and it may also cause objectionable noise in the pump. Air pressure is very useful around a powerhouse also for blowing out generators, for running small drills and jack hammers, and for small repair work. In units set below tailwater, compressed air has been used to depress tailwater below the runner to permit operation as synchronous condensers without undue friction; Rocky River, Connecticut, and Hiwassee, North Carolina, are examples.

A complete set of open-end wrenches, as well as any special box wrenches and sockets required for special work on the machinery, should be mounted near the units so as to be readily available. In addition, a number of jacks, bars, drills, and jack hammers should be on hand at a large powerhouse; smaller stations may not require them.

Inspection manholes, especially in concrete draft tubes and concrete spiral casings, are not usually furnished with the turbine but form an essential part of the installation. They should be rugged in design but not so cumbersome as to make removal for inspection too difficult.

Some plants at which the tailwater rises above the bottom of the turbine are provided with gates for the draft-tube outlet and pumps for pumping out the draft tube and casing, so that inspection and repairs can be made during high-tailwater periods if necessary. Usually, one pumping unit is sufficient, connections and valves being provided so that any draft tube can be pumped out at will.

Powerhouse cranes are considered a necessity in all except very small powerhouses. Their design is important and is discussed fully in Chapter 37.

The supply of continuous and adequate water for cooling oil bearings, or lubricating lignum vitae bearings, is important. The customary method is to tap the penstock or casing and filter the water through a twin strainer, either half of which can be cleaned without interfering with the operation of the other half. This is ordinarily considered satisfactory, but in cold climates where frazil ice forms, or where chips or leaves are troublesome at certain times of the year, such strainers have been known to clog up in a few minutes' time, and a number of lignum vitae bearings have been badly burned as a result. Revolving screens, heater tanks, and special intakes built into the dam have remedied this trouble at various plants.

22. Guide for Purchasers of Hydraulic Equipment. When asking for quotations on hydraulic turbines and accessories, the engineer should give the

manufacturers complete information regarding the physical characteristics of the site and the type of unit desired. The physical characteristics are

1. Maximum, minimum, and average static head.
2. Fluctuations in head and tailwater elevations.
3. Length and diameter of pipeline, if any, and preferably a profile of same.
4. The net head for which the turbines are to have the best efficiency.
5. The shaft horsepower desired at a given net head.
6. The speed at which the turbines are to operate.

The question of proper speed for the units should be definitely settled with the various water-wheel manufacturers by asking, sometime before calling for final prices, for their recommendations for the given conditions. It may be desirable to ask for both turbine and generator bids on two different speeds, so as to compare the cost of the units, together with their relative merits as to efficiency and the probability of pitting at the higher speed.

At the same time, the prospective purchaser may well obtain the manufacturers' advice as to the Wr^2 desirable in the generator and the allowable penstock velocities and spacing of units in the powerhouse. With this information at hand the prospective purchaser can issue a much more practical and more complete call for tenders and perhaps save both himself and the tenderers much time and expense.

The unit desired is usually described in a preliminary specification or call for tenders which gives in detail the type and characteristics of turbine and accessories. It is always desirable to consult one or more of the manufacturers before deciding on the type of unit, as their suggestions may save a considerable amount of time in arranging the details of the installation. The specifications should include

1. The type of wheel installation.
2. The percentage of full gate power at which it is desired to have the point of best efficiency.
3. The statement whether the most efficient type of unit is desired or whether efficiency is not of vital importance
4. The statement whether a reliable unit or a cheaper, less reliable one is preferred.
5. List of accessory equipment to be included, such as penstock valves, relief valves, steel draft tubes, governors, etc.
6. Limits to the turbine equipment, within which the manufacturer is to furnish all necessary parts.
7. Location of governor equipment.
8. A statement whether structural-steel supports are to be included. These are usually furnished by the purchaser.
9. Spare parts to be included, such as guide vanes, guide-vane links, bearings, lined bearing shoe (if water-lubricated), pump gears, regulator valve, etc.

The foregoing comments are in no way complete, and suggestions for additional information that should be included in the preliminary specifications are given subsequently in a "Form of Tabulation for Comparison of Hydraulic Equipment Quotations."

It is very difficult to obtain quotations that are strictly comparable unless complete preliminary specifications have been provided. To purchase from the lowest bidder without careful comparison of the specifications, accessories, details, and other influencing characteristics of the different quotations is the height of folly. Unless the preliminary specifications are sufficiently clear to make it certain that all bidders are quoting on the same class of apparatus, many conferences and revised quotations and much study will be necessary to line up all bids for comparison.

To assist the engineer in determining whether the manufacturers have included in the bids all the requirements of the specifications and in comparing the bids, the following form is given. He may find it useful also as a check list when preparing specifications.

FORM OF TABULATION FOR COMPARISON OF HYDRAULIC EQUIPMENT QUOTATIONS

General

1. Job.
2. Manufacturer.
3. Number of units.
4. Setting.
5. Casing.
6. Type of unit.
7. Power of each unit.
8. Head.
9. At turbine rating (7).
10. Average effective.
11. Maximum.
12. Minimum.
13. Direction of rotation.
14. Normal speed, rpm.
15. Runaway speed, rpm.
16. Specific speed.
17. Discharge diameter of runner.
18. Inlet diameter of runner.
19. Recommended vertical distance from bottom of runner to low tailwater.
20. Dates of delivery.
21. Parts for concrete.
22. Complete units.
23. Drawings of over-all dimensions.
24. Complete drawings.
25. Dimensions of unit affecting size of substructure.
26. Shop inspection.
27. Shop assembly.
28. Length of guarantees.
29. Shop painting.
30. Is field erection included in bid?
31. Daily rate of construction superintendent if field erection (30) is not included.

Weights and Price

32. Shipping weight.
33. Heaviest piece.

34. Total apparatus.
35. Weights of runner, shaft, and hydraulic thrust.
36. Total price and terms of payment.
37. F.O.B. where?

Efficiencies

38. Guaranteed efficiencies.
39. Full load.
40. _____% load.
41. _____% load.
42. Average between _____% and _____% load.
43. Tested efficiencies.
44. Full load.
45 _____% load.
46 _____% load.
47. Average between _____% and _____% load.
48. Where tested?

Details

49. Length and size of penstock reducers.
50. Wheel casing.
51. Type.
52. Material.
53. Thickness.
54. Drain pipes, fittings, and valves.
55. Manholes and material of bolts.
56. Draft-tube manhole and cover.
57. Turbine pit-liner material and length above center line of turbine.
58. Speed-ring material and construction.
59. Turbine gates.
60. Material.
61. Machined surfaces.
62. Clearances.
63. Sealing devices and estimated leakage at full head.
64. Stuffing boxes.
65. Method of lubrication
66. Removable wearing surfaces
67. Top of gate.
68. Bottom of gate.
69. Turbine-gate rigging.
70. Shifting-ring bearing linings.
71. Links designed to break.
72. Method of lubrication.
73. Individual adjustment.
74. Two or three bearings for gate stem
75. Runners.
76. Type.
77. Material.
78. How secured to shaft.
79. Capable of runaway speed?
80. Removable wearing surfaces.
81. At top of runner.
82. On runner.
83. On casing.

84. At bottom of runner.
85. On runner.
86. On casing.
87. Drainage back of runner.
88. Main shaft.
89. Material.
90. Polished.
91. Type of coupling.
92. Coupling bolts.
93. Coupling reamers.
94. Distance from center line of turbine to end of shaft.
95. Diameter.
96. Main-shaft bearings.
97. Type.
98. Lubrication.
99. Cooling system, piping, and screens.
100. Lining.

Governors

101. Type and number.
102. Capacity, foot-pounds and foot-pounds per second.
103. Servomotors or levers.
104. Number of servomotors and volume of each.
105. Oil pressure and quantity furnished.
106. Pumping system.
107 Pumping capacity in gpm each.
108. Piping and pressure tank size.
109. Hand control.
110. Switchboard control.
111. Guaranteed speed regulation.
112. Load-off to zero, time of closure.
113. ______% speed rise.
114. Load-on from zero.
115. ______% load, ______% speed drop.
116. Based on what generator Wr^2?
117. Dead beat.
118. Will not hunt.
119. Operate in parallel with others.
120. Sensitive to what percentage of load change.
121. Speed-limiting device.
122. Air compressor.
123. Water-pressure rise for full load off.
124. Water-pressure drop for full load on.

Accessories

125. Vacuum and pressure gages.
126. Tachometers.
127. Friction brakes.
128. Relief valves.
129. Breaking plates.
130. Penstock valves, companion flanges, and bolts.
131. Set of wrenches and wrenchboard.
132. Complete set of cup leathers, packing, etc.

133. All jigs, gages, and templates retained for replacements.
134. Handling devices.
135. Jack screws for breaking joints with bronze screw plugs.
136. Foundation bolts.

Workmanship

137. A comparison to be given here of workmanship specifications and working stresses used in design.

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7. "Guide Bearings for Vertical Hydro-Electric Units," National Electric Light Association publications 267–240, Edison Electric Institute, New York, June 1927.

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CHAPTER 40

ELECTRICAL DESIGN

By Raymond A. Hopkins *

1. Electrical Design. The electrical work of a hydroelectric development starts at the couplings which drive the generators and includes the generators, exciters, generator-voltage switching and connections, step-up transformers, transmission-voltage switching and connections, control switchboards and wiring, signals, annunciations, communications, auxiliary power supply and distribution, lighting, heating, and finally the transmission lines.

This chapter briefly reviews the fundamentals on which electrical design is based. For more detailed discussions, reference should be made to the bibliography at the end of this chapter.

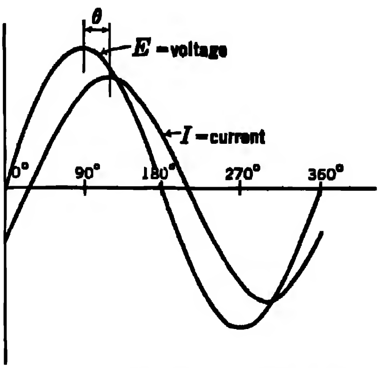

FIG 1. Current and voltage diagram in rectangular coordinates.

2. Current. A current of electricity is conceived of as a stream of moving electrical particles or quantities. The strength of current is measured by the number of unit quantities which pass a given cross-section of a conductor per unit of time. The unit of current is the ampere.

A direct-current circuit is one in which the direction of the current and voltage does not change. An alternating-current circuit is one in which the current and voltage alternate in direction periodically. With minor exceptions, electric energy is generated, transmitted, and utilized as alternating-current energy. In commercial power circuits the positive and negative loops are equal to each other and closely approach the sine-wave shape as illustrated by the graph in rectangular coordinates in Fig. 1.

3. Voltage. The flow of current in an electrical circuit is due to a difference of electric pressure or potential from point to point in the circuit. The total difference of potential applied to the circuit is called the impressed voltage. The unit of voltage is the volt.

The voltage of a circuit generally refers to the voltage between conductors, although the voltage from conductor to neutral or to ground may be less.

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The voltage between conductors is known as the line-to-line voltage, the voltage from a line conductor to the neutral of the system is known as the line-to-neutral voltage, and the voltage from a line conductor to ground is known as the line-to-ground voltage. When speaking of a system that has the neutral conductor carried out along with the line conductors so that either the line-to-line voltage or the line-to-neutral voltage is available at points of utilization, the system is referred to as a double-voltage system; examples: a single-phase, 3-wire, 220/110-volt system; a 3-phase, 4-wire, 4000/2300-volt system.

In order to allow for voltage drop in the distribution system, it is customary to rate generators 4 or 5% higher than motors, as, for instance, a 2300-volt generator and a 2200-volt motor.

Nominal voltages in general use in the United States, with those preferred by NEMA-NELA Joint Committee of 1930 italicized, are as follows:

Direct-current Voltages

12 24, 48—telephones, signals.
125, 250—lamps, motors, heaters, exciters.
550, 600—motors, heaters, railways.

Alternating-current Voltages

***115*, *220*, 230—lamps, motors, heaters**
***440*, 550—motors, heaters.**
***2300*, *4000*, 4600, 6600, 11,000, *13,200*—generators, distribution, motors**
22,000, *33,000*, 44,000 *66,000*, 110,000, *132,000*, 154,000, 187,000, *220,000*, *330,000* —transmission.

4. Effective Values. For alternating-current circuits, it is seen from Fig. 1 that the instantaneous values of current, I, and voltage, E, vary continually between maximum positive and negative values. The effective or root-mean-square (rms) value is the square root of the mean of the squares of the instantaneous values for one complete cycle. For a sine wave, the rms value is equal to the maximum value divided by $\sqrt{2}$. The rms value determines the energy and is always understood except when specifically stated otherwise.

5. Power. Electric power is the time rate at which electric energy or work is expended or delivered. The unit of power is the watt. For a simple resistance circuit, the power is equal to the current, I, multiplied by the voltage, E, or

$$P = EI \qquad [1]$$

6. Energy. Electric energy or work is expended in a circuit whenever a voltage is applied to the circuit and a current flows. The unit of energy is the watt-hour. For a simple resistance circuit, the energy is equal to the product of the current, I, the voltage, E, and the time in hours, t, or

$$W = EIt \qquad [2]$$

7. Frequency. The frequency of an alternating-current circuit is the number of cycles, or complete reversals, per second, which its current and voltage undergo.

A synchronous generator or motor operates at a fixed synchronous speed depending upon its number of poles and the frequency of the circuit. An induction generator or motor operates at a speed near synchronous speed, the slight difference being called the slip and depending upon the load. The relation between synchronous speed, number of poles, and frequency is given by the equation:

$$s = \frac{120f}{p} \qquad [3]$$

where s = synchronous speed in revolutions per minute;
f = frequency in cycles per second;
p = number of poles.

Various frequencies from 15 to 133 cycles have been used and a few are still in use. In North America, 60-cycle frequency is standard, although lower frequencies still exist in a few cases. In southern California a large 50-cycle system has recently been changed to operate at 60 cycles. Around Niagara Falls, considerable energy is generated and used locally at 25 cycles. Outside North America the standard is generally 50 cycles, with some railway and industrial use at $16\frac{2}{3}$ cycles. In general, 60-cycle frequency is preferred because it avoids flicker in lighting and requires less expensive transformers. The lower frequencies are, however, slightly more favorable for transmission and for large rotating machinery.

8. Number of Phases. The phase of an alternating-current circuit or device usually refers to the number of phases, as single-phase, 3-phase, polyphase. A further explanation of these terms is given in Chapter 41, Section 1, Generators. Fundamental voltage, current, and power relations in 1-, 2-, and 3-phase systems are indicated in Fig. 2. Comparative voltages, currents, conductor sizes, and losses are given in Table 1.

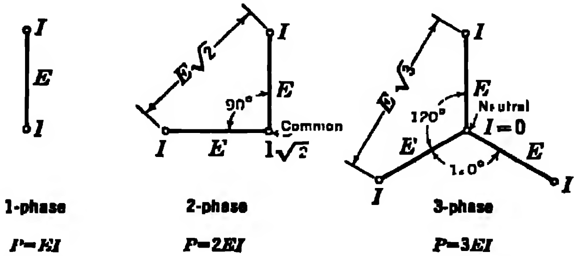


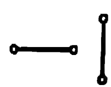
FIG. 2. Voltage, current, and power relations in balanced 1-phase, 2-phase, and 3-phase circuits.

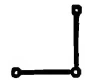
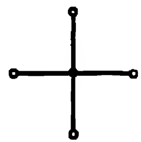
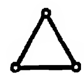
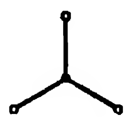
Single-phase circuits are used extensively for lighting, heating and general utility in buildings. They also are used for extending polyphase distribution lines as in rural distribution and for some railway electrifications where series motors are used. For induction and synchronous motors, however, the single-

TABLE 1

Comparison of Distribution Systems

Based on equal lengths of line and equal balanced kva loads Single-phase, 2-wire, 100-volt system used as basis of comparison Figures are rounded off

System	Diagram	Current per Conductor	Conductor Size Based on Heating of Conductor (Short Line)				Conductor Size Based on Per Cent Voltage Drop (Long Line)			
			Conductor Size	Total Conductor Material	Per Cent Voltage Drop	Power Loss	Conductor Size	Total Conductor Material	Per Cent Voltage Drop	Power Loss
A. 1-ph, 2-wire 100-volt		100	100	100	100	100	100	100	100	100
B. 1-ph, 3-wire 200/100-volt		Line 50 Neut 0	Line 30 Neut. 25 *	62	50	50	Line 25 Neut 12 *	31	100	100
C. 2-ph, 4-wire 100-volt		50	50	100	100	100	50	100	100	100

D. 2-ph, 3-wire 141/100-volt		Line 50 Comm 71	Line 50 Comm 71	85	100	85	Line 50 Comm. 71	85	100	85
E. 2-ph, 5-wire 200/100-volt		Line 25 Neut 0	Line 25 Neut 12 *	56	50	50	Line 12 Neut. 6 *	28	100	100
F. 3-ph, 3-wire 100-volt		58	58	87	87	87	50	75	100	100
G. 3-ph, 4-wire 173 100-volt		Line 33 Neut 0	Line 33 Neut 17 *	58	50	50	Line 17 Neut 8 *	29	100	100

* Neutral carries no current under balanced load conditions but usually is proportioned according to expected amount of unbalance. For this comparison the total conductor material is based on the neutral conductor size shown, but voltage drop and power loss are based on balanced loads with no current in the neutral.

phase circuit is not desirable since it has no definite phase rotation. The single-phase motor must have a starting, winding, and centrifugal switch to disconnect it after starting, or some other form of starting device. Only small motors are usually so built.

Polyphase circuits carry the phase rotation of the generators. This produces a rotating field in the induction and synchronous motor which causes it to start and gives it a definite direction of rotation. For this reason the polyphase circuits are used almost exclusively for all power transmission and distribution. Two-phase systems have the disadvantages of many wires as in systems C and E, Table 1, or of dissymmetry as in system D, and although a few 2-phase systems are still in existence they are being replaced as fast as economically possible. The 3-phase system is practically universal for new work, the 3-wire system being used for all power applications and the 4-wire system being used where single-phase, lower-voltage loads are expected.

The higher-voltage systems, B, E, and G of Table 1, require less conductor material than the others, even when the neutral conductor is included, and considerably less if the neutral conductor is not required. In most cases the power loss is also less. Insulation need not be increased if the system neutral is grounded.

9. Circuit Constants. Every electric circuit is characterized by one or more of the constants resistance, inductive reactance, and condensive reactance, which are independent of current and voltage but depend upon the physical composition and dimensions of the circuit. The resistance determines the energy loss by heat; the inductive reactance determines the energy storage in the surrounding magnetic field; and the condensive reactance determines the energy storage in the surrounding dielectric field. The three constants are defined more specifically by the following equations:

$$R = \frac{\rho l}{a} \qquad [4]$$

$$X_L = 2\pi f L \qquad [5]$$

$$X_C = \frac{1}{2\pi f C} \qquad [6]$$

where R = resistance in ohms;
X_L = inductive reactance in ohms;
X_C = condensive reactance in ohms;
ρ = resistivity in ohms per mil-foot;
l = length in feet;
a = cross-sectional area in circular mils;
π = 3.1416;
f = frequency in cycles per second;
L = inductance in henries;
C = capacity in farads.

When a circuit contains resistance only, the current is in phase with the voltage, as in Fig. 3. If it contains inductive reactance only, the current lags the voltage by 90 degrees as in Fig. 4. If it contains condensive reactance only, the current leads the voltage by 90 degrees, as in Fig. 5. Usually a circuit contains all three constants to some extent, and the amount the current lags or leads the voltage depends on the relative values of the inductive and condensive reactances. A circuit of this last type is represented in Fig. 1, where the current lags the voltage by θ degrees. For convenience, the difference between the inductive and condensive reactances, which can be considered the net reactance, is called simply the reactance and is designated by the letter X. The inductive reactance is hereinafter considered positive and the condensive reactance negative. The reactance may be either positive or negative, depending on which one of its two components has the larger value; thus:

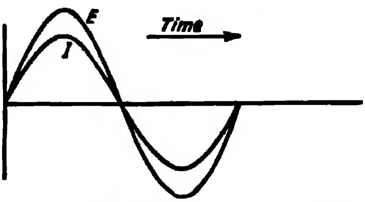


FIG. 3. Circuit containing resistance only.

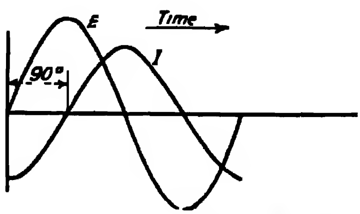


FIG. 4. Circuit containing inductive reactance only.

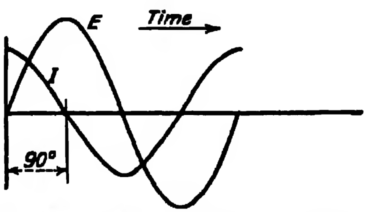


FIG. 5. Circuit containing condensive reactance only.

$$X = X_L - X_C \qquad [7]$$

The resistance, R, and the reactance, X, of a circuit, being 90 degrees out of phase with each other as regards their effect on the current, may be resolved into one equivalent value called the impedance, Z, thus:

$$Z = \sqrt{R^2 + X^2} \qquad [8]$$

The voltage across the terminals of a circuit in terms of the current and the circuit constants is, from Ohm's law:

$$\left.\begin{aligned} E &= IZ \\ &= I\sqrt{R^2 + X^2} \\ &= \sqrt{(IR)^2 + (IX)^2} \\ &= \sqrt{E_1^2 + E_2^2} \end{aligned}\right\} \qquad [9]$$

where E = voltage in volts;
I = current in amperes;
Z = impedance in ohms;
R = resistance in ohms;
X = reactance in ohms;
$E_1 = IR$ = power component of E;
$E_2 = IX$ = reactive component of E.

Likewise, the current may be expressed in terms of the voltage and reciprocals of the circuit constants as follows:

$$\left.\begin{aligned} I &= EY \\ &= E\sqrt{G^2 + B^2} \\ &= \sqrt{(EG)^2 + (EB)^2} \\ &= \sqrt{I_1^2 + I_2^2} \end{aligned}\right\} \qquad [10]$$

where I = current in amperes;
E = voltage in volts;
$Y = 1/Z$ = admittance in mhos;
$G = R/Z^2$ = conductance in mhos;
$B = X/Z^2$ = susceptance in mhos;
$I_1 = EG$ = power component of I;
$I_2 = EB$ = reactive component of I.

Resistance, reactance, and impedance are usually more convenient for use with series circuits, while conductance, susceptance, and admittance are more convenient where parallel circuits are involved.

10. Power Factor and Reactive Factor. The cosine of the angle by which the current leads or lags the voltage is called the power factor of the circuit. The power (kilowatts) is the product of the current, voltage, and power factor, thus:

$$P = EI \cos\theta \qquad [11]$$

The sine of the angle by which the current leads or lags the voltage is called the reactive factor of the circuit. The reactive kilovolt-amperes is the product (kilovars) of the current, voltage, and reactive factor thus:

$$Q = EI \sin\theta \qquad [12]$$

The kilovolt-amperes, or kva, of a circuit is the product of the current and voltage, thus:

$$K = EI \qquad [13]$$

11. Vector Representation. Alternating currents and voltages are conveniently represented by vector diagrams, as illustrated in Fig. 6, which

represents the same currents and voltages as are represented in rectangular coordinates in Fig. 1. The length of the vector corresponds to the effective value of the current, or voltage, and the angular position represents its phase displacement with reference to the axes. The vector rotation is considered as counterclockwise, so that in Fig. 6, as in Fig. 1, the voltage leads the current by the angle θ. Each vector has two components, one along the X axis, the other along the Y axis. Components measured to the right of YY or above XX are positive, while those measured to the left of YY or below

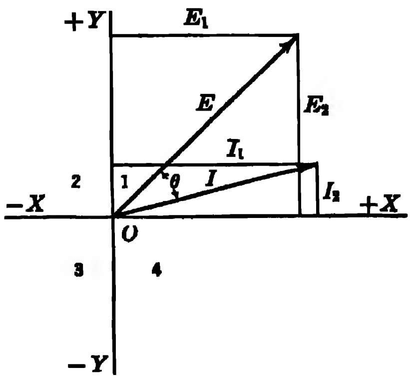


FIG. 6. Current and voltage vector diagram.

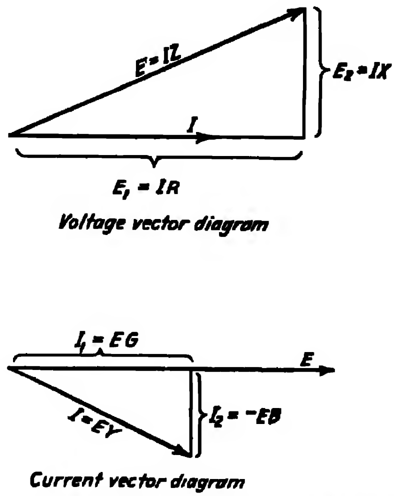


FIG. 7. Vector component diagrams.

XX are negative. For convenience, the four quadrants are always numbered in the order shown in Fig. 6. Two or more vector quantities may be added or subtracted by adding or subtracting their components. The relations given analytically in Section 9 are shown graphically in Fig. 7.

12. Complex Algebra. Calculation of the performance of electrical circuits involving vector quantities is greatly facilitated by the use of complex algebra, the principles of which are covered in most textbooks on algebra, and very completely in Dr. C. P. Steinmetz' *Engineering Mathematics*. Equation 8 is expressed in complex notation in the following form:

$$\underset{\cdot}{Z} = R + jX \tag{14}$$

The dot under the Z indicates that Z is a vector quantity having two components at right angles to each other, namely R and X. The R part of the right-hand member is measured along the X axis and is called the real part of the quantity. The X part, since it is preceded by the symbol j, is measured along the Y axis and is called the imaginary part of the quantity.

The symbol j, in addition to indicating measurement along the Y axis, also has the value $\sqrt{-1}$, or $j^2 = -1$. Addition, subtraction, multiplication, and division of complex quantities are performed in the same manner as with ordinary algebraic quantities, by observing this twofold significance of the symbol j.

13. Short-circuit Analysis. Analyses of the short-circuit characteristics of a power system are essential for determining (1) the interrupting duty

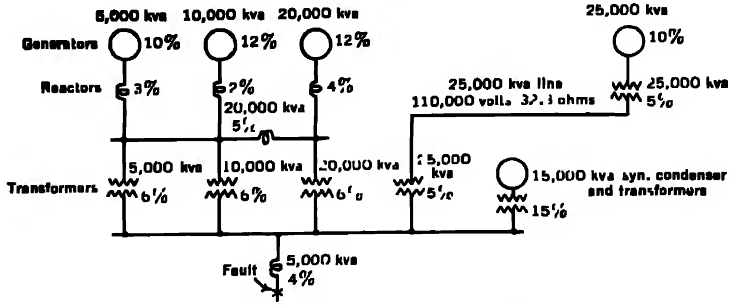


FIG. 8. System reactance diagram.

for which the circuit breakers must be designed, (2) the mechanical stresses which the insulators must withstand, (3) the thermal and mechanical capacities required for current transformers and other series equipment, (4) proper relay settings, and (5) other, similar studies. The value of short-circuit current due to a fault, that is, a short circuit between phases or from phase to ground, at any point on a system depends on (1) the connected capacity in synchronous generators and motors, (2) impedances of the synchronous machines, and (3) impedances of transformers, reactors, and circuits between the machines and the fault.

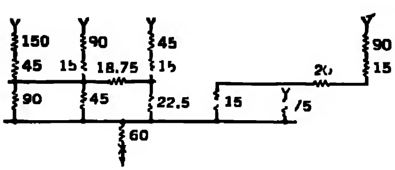


FIG. 9. Equivalent reactance diagram. (All reactances in percentages to 75,000-kva base.)

The analytical solution of a single-phase or a balanced 3-phase network, where resistance is neglected and reactances are assumed in phase with each other, consists of replacing element after element by equivalent elements of simpler construction so as to resolve the system finally into one reactance connecting the synchronous equipment with the fault. A typical system is shown in Fig. 8. The reactance values used apply to one conductor of the 3-phase circuit.

The first step is to replace the generators, motors, reactors, transformers and circuits by their equivalent reactances, giving Fig. 9. All reactances must be expressed in the same terms, that is, either in ohms at a common voltage base, or in percentages to a common kva base. When the latter

method is used the common base is usually arbitrarily chosen as 100,000 kva, or as the total connected kva synchronous capacity. The following relations are useful in reducing the various elements of the system to a common base:

$$\frac{\text{Reactance in percentage}}{\text{Reactance in ohms} \times 100} = \frac{\text{Rated volt-amperes}}{(\text{Rated volts})^2} \tag{15}$$

$$\frac{\text{Reactance in percentage to base } K_1}{\text{Reactance in percentage to base } K_2} = \frac{K_1}{K_2} \tag{16}$$

$$\frac{\text{Reactance in ohms at voltage } E_1}{\text{Reactance in ohms at voltage } E_2} = \frac{E_1^2}{E_2^2} \tag{17}$$

Subsequent steps in reducing the network are shown in Fig. 10. The working tools are explained by the elementary diagrams of Fig. 11. Two

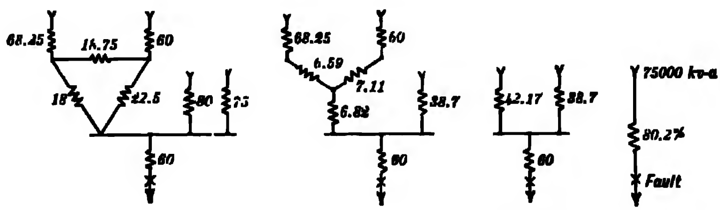


FIG. 10. Simplification of typical network

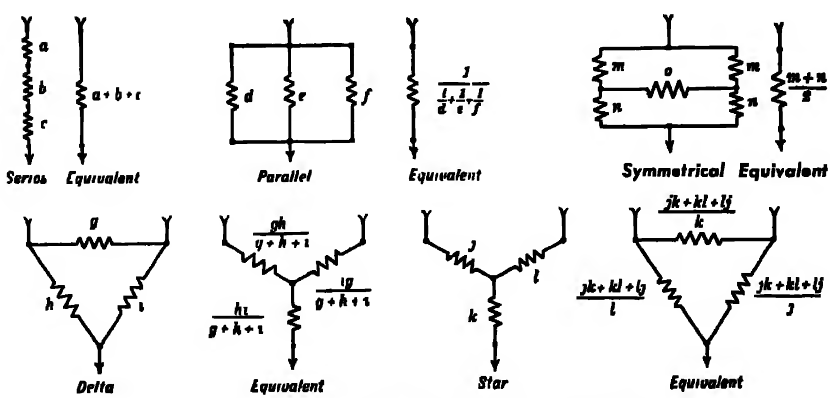


FIG. 11. Elements of simplification of networks.

very useful rules for reducing networks and avoiding the use of reciprocals are as follows:

1. Series reactances may be combined by reducing to the same base and adding percentages.
2. Parallel reactances may be combined by reducing to the same percentage and adding bases.

Where these simple substitution processes are not adequate to reduce the network to a single reactance, it becomes necessary to set up a series of differential equations based on Kirchhoff's laws and solve them by substitution. For analytical methods, see Section 14.

By dividing the rated current per conductor by the percentage reactance, found as described above and expressed as a decimal, the initial value of short-circuit current is obtained. If the reactance has been worked out in ohms, the initial value of short-circuit current can be obtained by dividing the rated voltage to neutral by the ohms. The value obtained by either method is known as the initial rms symmetrical short-circuit current per conductor. If the short circuit occurs at the instant the voltage wave is passing through zero, the short-circuit current wave will be offset about its axis, the initial rms unsymmetrical current being approximately 1.73 times the symmetrical current. The armature reaction of the synchronous machines rapidly reduces the initial value of short-circuit current, the final, steady value being known as the sustained short-circuit current.

For the application of oil circuit-breakers, advantage is taken of the fact that the short-circuit current decreases during the first few seconds after the instant of short circuit, and hence the value of current which the breaker will be called upon to interrupt will depend not only on the initial value of short-circuit current but also on the time elapsed between the short circuit and the parting of the breaker contacts. As the result of innumerable tests and oscillographic records, the manufacturers of synchronous machines have obtained current-decrement factors of standard machines, which are given in Fig. 12. The value of rms unsymmetrical short-circuit current in amperes at moment of opening, which is the basis of circuit-breaker rating, is obtained by multiplying the rated current of the system, corresponding to the kva base at which the reactance of the system is expressed, by the factor taken from the curve. In applying the decrement factors, any branches of the system that feed directly into the fault should be treated separately. For instance, in Fig. 8, for a fault occurring on the bus instead of on the feeder beyond the reactor, the entire system should be treated as three separate branches: (1) the 5000-, 10,000-, and 20,000-kva generators; (2) the 25,000-kva generator, and (3) the 15,000-kva condenser. Each of these branches should be reduced to a single reactance based on the kva of the branch, and, after the decrement curves have been applied to each, the resulting three short-circuit currents should be added to obtain the total short-circuit current into the fault.

The analysis outlined above is intended to cover 3-phase short circuits or single-phase line-to-line short circuits. As the latter have less decrement than the former they represent a more severe circuit-breaker duty and are used for the selection of breakers. For this reason the curves of Fig. 12 are drawn to apply to single-phase, line-to-line short circuits. Under some conditions of neutral grounding, the single-phase, line-to-neutral short circuit

is even more severe than the single-phase line-to-line and should be investigated [10, 12].

For the determination of the mechanical forces to which bus supports and insulators will be subjected when a short circuit occurs, consideration must be given to the initial peak unsymmetrical value of current in at least one phase.

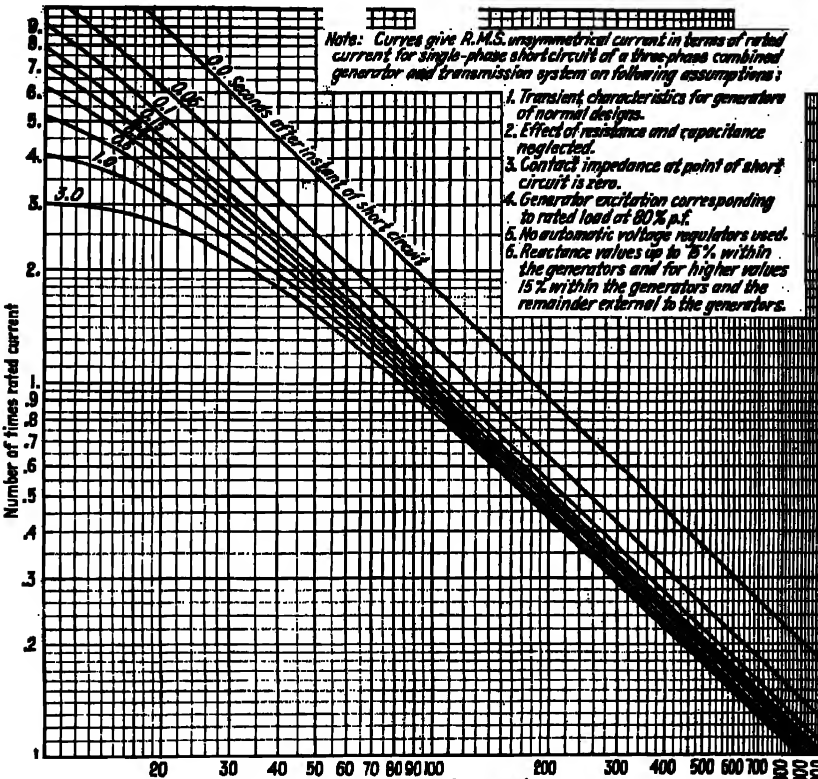


FIG. 12. Short-circuit current-decrement curves.

The generally accepted value is equal to 1.8 $\sqrt{2}$ times the initial rms symmetrical value. In equations 19, 20, and 21 the rms symmetrical values are used, but the effect of the peak unsymmetrical current in one phase is included in the constants. The notation is as follows:

F = force, in pounds per foot of conductor. The force between two conductors is attraction when the currents are in the same direction, and repulsion when in opposite directions;

S = spacing between centers of conductors in inches;

$i_1 i_2$ = simultaneous values of currents in the two conductors;

I_0 = initial rms symmetrical short-circuit current in amperes.

The fundamental equation giving the force acting between two parallel conductors is:

$$F = 5.4 \frac{i_1 i_2}{S \times 10^7} \tag{18}$$

For a single-phase, line-to-line short circuit on a single-phase system, or on a three-phase system with conductors arranged in any configuration, the force is repulsion acting in the plane of the two conductors under short circuit. The maximum value is:

$$F = 35 \frac{I_0^2}{S \times 10^7} \tag{19}$$

For a 3-phase, line-to-line short circuit on a 3-phase system with conductors arranged at the vertices of an equilateral triangle, the maximum force acts away from the center of the triangle and is:

$$F = 30.3 \frac{I_0^2}{S \times 10^7} \tag{20}$$

For a 3-phase, line-to-line short circuit on a 3-phase system with conductors arranged in a plane, the forces act in the plane of the conductors and the maximum value of the force on any conductor is:

$$F = 26.3 \frac{I_0^2}{S \times 10^7} \tag{21}$$

Equations 19, 20, and 21 are commonly used by applying a factor of safety of 2 and specifying that the insulators shall withstand the resulting stress when applied in any direction through the center of the conductor clamp. For a discussion of the effect of shape of conductor at very close spacing see Section 17, Ref. 9. For a discussion of the effect of the natural frequency of the conductors and the supports under certain critical conditions, see Section 17, Ref. 11.

14. Analytical Methods. Two powerful analytical methods of attacking problems involving balanced or unbalanced conditions in networks are the method of symmetrical components and the tensor analysis method [12, 17].

15. Network Analyzer. The analysis of the performance of power system networks by mathematical calculation alone is practicable only for comparatively simple systems. When the network becomes complicated, as it does with nearly all major power systems, some other method must be used. One of the most useful devices is the alternating-current network analyzer. It consists of a number of small, adjustable resistors, reactors, and condensers arranged on a switchboard with facilities for interconnecting them with flexible cords so that by proper adjustments and connections a miniature of the system under study can be set up and tested. Generators are represented by small phase-shifting transformers. Power is secured from a small motor-generator set, and instruments are provided which can be plugged in by suitable keys to read current and voltage at various points in the sys-

tem under various conditions of loading and of fault that may be imposed. Studies may be made of circuit-breaker duty, relay requirements, load division, voltage regulation, and static and transient stability. The accuracy is well within the limits of the known constants of the actual system.

A somewhat similar, but much simpler, direct-current calculating board employing resistors only may be used to great advantage for simpler problems of network solutions where phase angles can be neglected and only current magnitudes are involved [19, 21].

16. Power-system Stability. A power system consisting of a group of synchronous generators, a system of transmission lines, and a group of resistance, induction, and synchronous loads will deliver steady load successfully up to a certain critical value, at which the torque in the air gap of one of the synchronous machines is no longer adequate and the machine is pulled out of step. The system is said to be in steady-state stability at steady loads below the critical value and to be unstable at greater loads. Again while the system is carrying load below its critical steady load, a sudden change of load, or a fault or other transient disturbance, may cause one of the synchronous machines to swing beyond its ability to recover and finally to be pulled out of step. The system is said to be in transient stability at those loads or for those transients within which it can regain its equilibrium, and to be unstable at greater loads or for greater transients. Power-system stability is, therefore, that quality which enables the system to carry its load successfully in equilibrium under steady load conditions or to regain its equilibrium after a transient disturbance tending to pull it out of synchronism.

Factors known to improve the stability of the usual power system include: low reactance in generators, transformers, and circuits; bussing of machines, transformers, and lines; resistance or reactance in the grounded neutral connections of transformer banks; high-speed relaying and breaker tripping; ample reserve of spinning generator capacity well scattered over the system near the loads; flywheel effect in rotating machinery; low-resistance damper windings on synchronous machines; quick-response excitation; and prevention of lightning disturbances on transmission lines by providing overhead ground wires, low tower-to-ground resistance, and high conductor insulation [15, 20].

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CHAPTER 41

GENERATORS, EXCITERS, AND TRANSFORMERS

By Raymond A. Hopkins *

1. Generators. A generator is defined as a machine that transforms mechanical energy into electrical energy. The essential features are (1) a field or assembly of magnets arranged to produce a magnetic flux, and (2) an armature or assembly of electric conductors arranged across the path of the magnetic flux. The field and armature are so mounted that by the application of mechanical force a relative motion is produced between the magnetic flux and the electric conductors, and this motion induces in the conductors an electromotive force. Since the field poles are arranged alternately, positive and negative, around the periphery of the generator, the polarity of the electromotive force induced in the armature is alternating.

The alternating-current synchronous generator, or alternator, delivers its induced alternating current directly to the external circuit. Alternating-current generators are used in hydroelectric stations where the output is to be transmitted over long lines, since the alternating current can be transformed up to suitable transmission voltage. They are also generally used where the output is to be distributed locally, since the greater part of power and lighting service is alternating.

The single-phase generator has one armature winding arranged to deliver a single-phase alternating current to a 2-wire system. The center point of the winding is sometimes brought out to a third terminal and can be used as a grounding point or as a neutral for a 3-wire system if desired. Single-phase generators are only occasionally used to supply special loads, as for single-phase railways and electrochemical and electrothermal industries, and in smaller sizes for lighting loads. The arrangement of the single-phase cores and windings is inherently uneconomical, with the result that a single-phase generator is about 50% larger than a 2- or 3-phase generator of equal capacity.

The 2-phase generator has two armature windings arranged to deliver two single-phase alternating currents, 90 electrical degrees apart in phase, to a 4-wire system; hence the machine is sometimes called a quarter-phase generator. The center points of the two windings are sometimes connected together, and then the generator is said to be interconnected. The point of

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interconnection may be used as a grounding point or as a neutral for a 5-wire system if desired. If the machine is not interconnected, two of the terminals, one from each winding, may be connected together so that the machine can be used on a 3-wire system. The common connection can be grounded if desired. Two-phase generators are used only in territories where 2-phase systems are established, but, owing to the better economy and performance of 3-phase generators and transmission systems, it often is preferable to use 3-phase generators even if phase converters are necessary.

The 3-phase generator has three armature windings arranged to deliver three single-phase alternating currents, 120 electrical degrees apart in phase, to a 3-wire system. The delta-connected, 3-phase generator has the end of each winding connected to the beginning of the next winding, the three points of connection being used as line terminals. The star or Y-connected machine has one end of each winding connected together in common, the other three ends being used as line terminals. The common connection may be used as a grounding point or as a neutral for a 4-wire system if desired. The delta connection is sometimes used for low-voltage, high-amperage machines, but usually the star winding is preferred, because of the higher voltage between terminals as compared with the voltage of each winding and because of the better opportunity to apply relay protection. Three-phase generators are always preferred to 2-phase or single-phase machines except where the latter are needed to meet the local conditions mentioned above.

The interconnections between windings, described above, are sometimes made within the machine by the manufacturer, but with the larger machines it is usual for the manufacturer to bring out both ends of each winding to terminals. The interconnections are then made in the station and are arranged to include the necessary current transformers for relay protection.

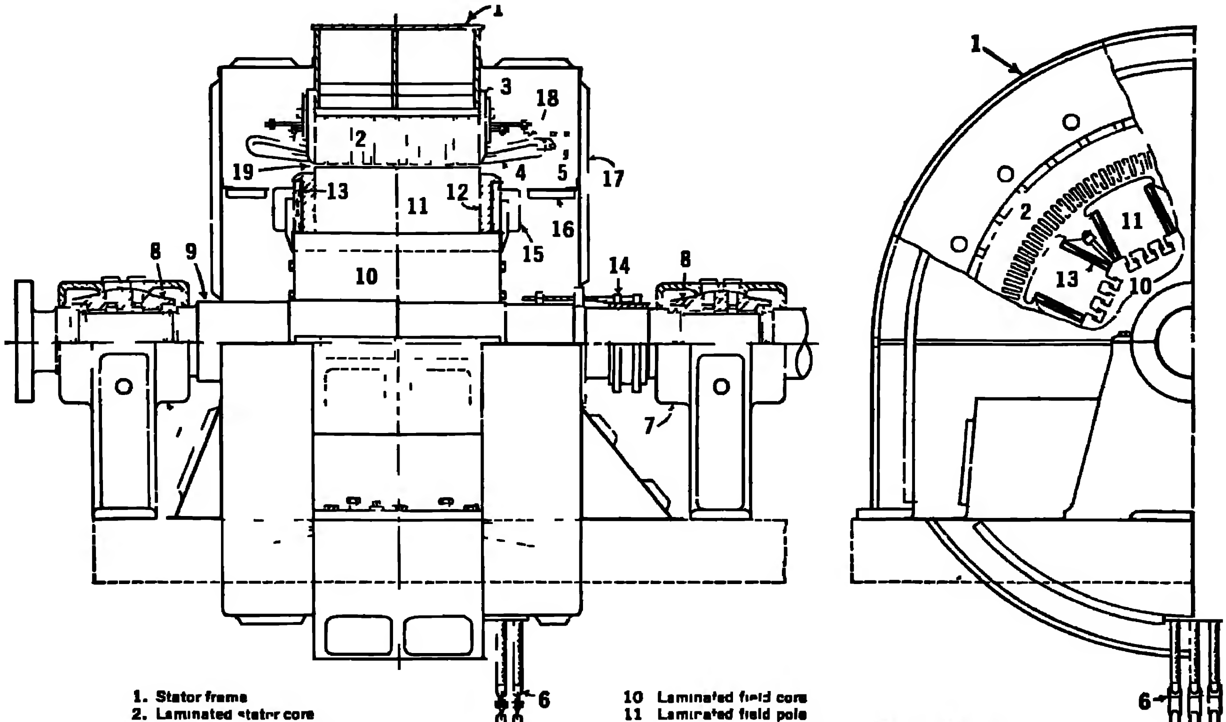
Induction motors occasionally are driven slightly above synchronous speed and operated as generators, but these instances are very rare. For information on induction generators, see Chapter 40, Section 17, Refs. 5 and 7.

The direct-current generator is provided with a commutator which rectifies its induced alternating current and delivers direct current to the external circuit. Direct-current generators are occasionally used where the output is to feed local railway or industrial loads which require direct current. Small direct-current generators, called exciters, are required in all power stations for energizing the magnetic fields of the alternating-current generators.

2. Generator Construction. Cross-sectional views showing the principal parts of a moderate-speed, horizontal generator and a low-speed, vertical generator are presented in Figs. 1 and 2

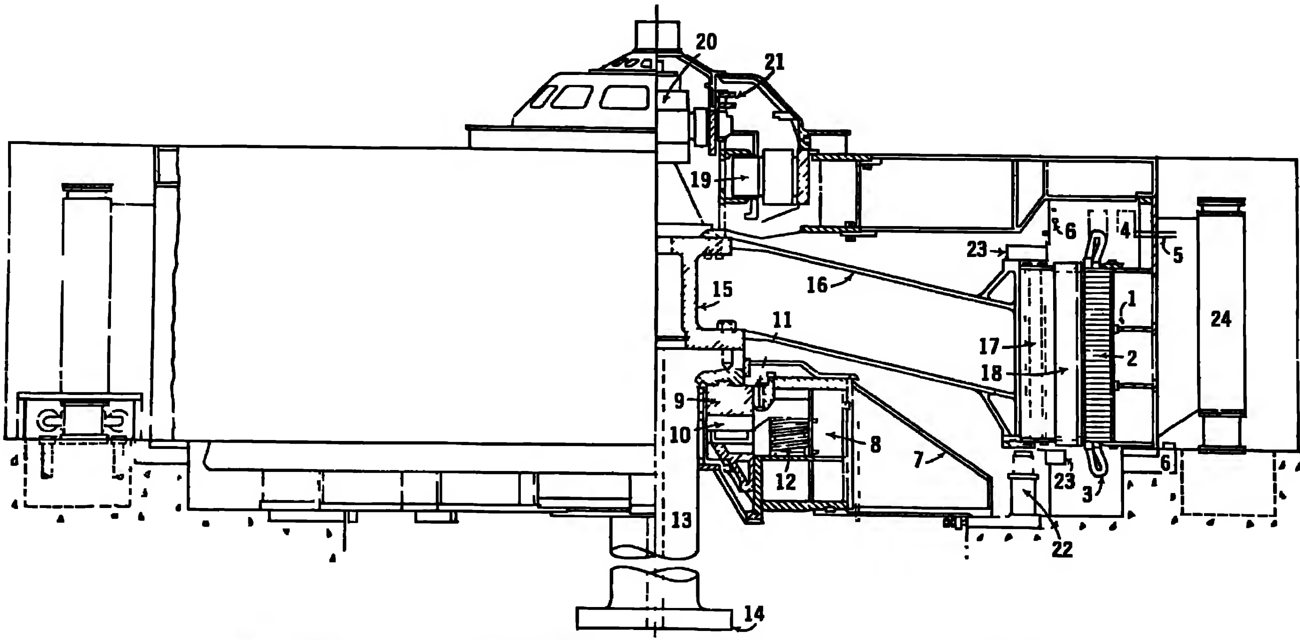
Horizontal-shaft and vertical-shaft generators are essentially the same electrically, merely being modified mechanically to suit the particular type of construction. The position of the shaft generally is chosen so as to obtain the most favorable turbine design.

• The revolving-field type has become practically standard for hydro generators for the following reasons: (1) the armature windings, which carry



1. Stator frame
2. Laminated stator core
3 Stator clamping flange
4 Stator winding
5. Stator end connections
6. Stator terminals
7. Bearing pedestals
8 Bearings
9 Shaft
10 Laminated field core
11 Laminated field pole
12. Field pole and plates
13 Field winding
14 Collector rings
15. Fans
16. Air deflectors
17 End shields
18. Stator winding supports
19 Air gap

FIG 1 Sectional view of typical horizontal water-wheel-driven generator (General Electric Co.)



1 Stator frame
2. Laminated stator core
3 Stator winding
4 Stator end connections
5 Stator leads
6 Fire extinguisher piping
7 Bearing bracket
8 Oil reservoir
9 Thrust runner
10 Thrust-bearing shoes
11. Guide bearing shoes
12 Bearing oil-cooling coils
13 Shaft
14 Coupling flange
15 Rotor hub
16 Rotor spider
17 Laminated field core
18 Laminated field coils
19 Main exciter
20 Pilot exciter
21 Rotor slip rings
22 Brake and jack
23 Rotor fans
24 Air cooler

FIG 2 Sectional view of typical vertical water-wheel-driven generator (Westinghouse Electric Corp.)

heavy currents and must be insulated for relatively high voltages, are freer from mechanical strains and vibrations when built in the form of a stator, while the low-voltage field windings are more adaptable to rotor construction; (2) the problem of taking off the large, high-voltage currents from a rotating armature would be very serious, but the lighter field currents can be handled very satisfactorily by slip rings and brushes.

The *stator* consists of a cast-iron or fabricated-steel frame, a laminated magnetic core, and the armature windings. The windings are form-wound and laid in slots in the core. Large machines generally are wound with several parallel circuits for each phase. The phases usually are connected in delta. Both ends of each phase winding generally are brought out to terminals so that differential relay protection can be provided. Frequently, in modern practice, both ends of two separate groups of parallel circuits are brought out, and split differential relay protection is applied. The coil surfaces are treated to prevent corona discharge, and the ends of the coils projecting beyond the core are braced to prevent distortion under short circuit. Resistance-type temperature detectors usually are built into the winding at a number of places to measure the operating temperatures. For sizes small enough to ship in one piece, the stator is assembled completely and wound in the factory. In larger sizes the stator is built in two or more sectors which are assembled and wound in the factory except for certain coils which span the splits, these being installed after assembly in the field. For very large sizes, the laminated core is stacked and all coils are installed after assembly of the frame in the field. The stator frame of the vertical generator rests on sole plates grouted into the masonry of the powerhouse structure while that of the horizontal generator usually rests on a bed plate which also carries the bearing pedestals.

The *rotor* is of the salient-pole type with windings surrounding the field poles which are dovetailed to the periphery of the laminated core. For low-speed rotors, the laminated core consists of a floating-type rim mounted on a cast or fabricated spider with hub attached to the shaft. For medium- and high-speed rotors, the core usually is mounted directly on the shaft. The pole windings are connected in series, and terminals are extended to slip rings located so that brushes can be inspected during operation.

The *shaft* usually is of forged steel, heat-treated, finished all over, and, in large sizes, bored axially to assist inspection of the forging. For a horizontal generator driven by a single turbine, one end of the shaft usually is flanged to couple to a similar flange on the end of the turbine shaft. If driven by two turbines, the shaft generally is extended at both ends and the two turbine runners are mounted directly on the two ends of the shaft. For a vertical generator, the shaft is flanged at the bottom end to couple to the turbine shaft. A single shaft sometimes is used for a vertical unit, particularly if the generator and turbine are built by the same manufacturer and if being in one piece does not increase the necessary crane height. The rotor sometimes is shrunk and keyed to the shaft. A design recently used

extensively in large vertical units provides at the top end of the shaft an integral flange which rests on the thrust bearing and to which the rotor is bolted. To insure accurate alinement of large coupled shafts, it is common practice to assemble the generator shaft and the turbine shaft in the manufacturer's shop to test perpendicularity between the face of the thrust bearing runner and the axis of the guide bearing journals.

The *coupling* of a large vertical unit is a very important detail, especially in a design having only two guide bearings, one at the generator and the other at the turbine. One practice uses body-bound bolts which must be shrunk when inserted or withdrawn. Another uses snug-fit bolts which can be withdrawn without shrinking. In either case, with the two coupling flanges matched together, the bolt holes are reamed and the bolts ground individually to insure accurate fits. In tightening the bolts, they usually are stretched a predetermined amount so that the aggregate tension exceeds the maximum vertical load to be carried by the coupling. In removing the bolts, a jack usually is required.

Bearings are universally babbitt-lined, oil-lubricated. Horizontal generators always have two main bearings. Vertical generators are provided with one thrust bearing to take the vertical load of the rotating parts of the generator and the turbine as well as the vertical hydraulic thrust, and with one or two guide bearings. Some form of thrust bearing must be used for a horizontal-shaft unit to resist hydraulic unbalance, which cannot be eliminated entirely. Sections 7 and 8 of Chapter 39 discuss thrust bearings and guide bearings.

A heavy *bearing bracket* is required for the vertical generator to carry the thrust bearing load down to the foundation. With the thrust bearing above the rotor, the bracket rests on the stator frame. If the thrust bearing is below the rotor, the bracket spans the turbine pit just below the stator. Two general forms of bearing bracket are common. One consists of two heavy, parallel beams with access to the bearing through the sides of the square oil reservoir. The other consists of several radial arms and a large hub containing the oil reservoir with provision for lowering the bearing for maintenance.

Brakes usually are provided on a water-wheel-driven generator to bring it to rest after being shut down. Without brakes slight leakage through the gates might cause the machine to rotate indefinitely. On a horizontal machine the brake consists of a band or two shoes bearing against a flanged flywheel. On a vertical machine several brake shoes bear against a machined surface on the bottom of the rotor. The braking force may be applied by magnets, oil, water, air, or purely mechanical means. The usual medium is air, since it usually is available in the station, gives a smooth braking effect due to its compressibility, and distributes the braking effect uniformly among the various shoes. With a vertical machine the brake cylinders can be used with air for braking and with oil for jacking up the rotor. The brakes usually are controlled manually from a point near the machine. They can be arranged to be applied automatically if required, and this usually is done in

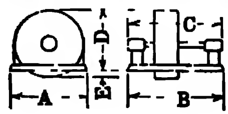
automatic stations. When stopping a large generator the rotor usually is allowed to decelerate to about 50% speed by windage resistance before the brakes are applied.

3. Weights and Dimensions. Approximate weights and dimensions of typical hydro generators of normal design are given in Tables 1 and 2. The

TABLE 1

APPROXIMATE WEIGHTS AND DIMENSIONS HORIZONTAL WATER-WHEEL GENERATORS *

This table applies to 3-phase, 60-cycle, 0.8-pf generators without direct-connected exciters.



ing, kva	Speed, rpm	Effect, lb-ft²	Weight,† lb	A	B	C	D	E
1,000	600	10,000	16,000	83	56	70	64	10
	300	53,000	23,000	101	62	80	72	18
	150	216,000	33,000	140	67	90	99	28
2,000	600	20,000	24,000	101	68	86	72	18
	300	107,000	33,000	131	74	97	95	24
	150	433,000	47,000	179	74	101	117	41
5,000	514	113 000	53,000	138	95	114	72	46
	300	328,000	75,000	173	98	119	90	60
	150	1,320,000	117,000	238	103	142	124	72
12,500	514	405,000	122,000	157	133	157	81	55
	300	1,130,000	155,000	189	141	168	98	66
	150	4,220 000	285,000	270	146	179	140	94
18,750	514	720,000	177,000	173	158	189	90	60
	300	1,940,000	228,000	217	158	189	113	75
	200	4,190,000	320,000	270	149	180	140	94
25,000	450	1,360,000	240,000	189	161	190	98	66
	300	2,850,000	300,000	216	166	199	113	75
31,250	450	1,850 000	300,000	189	170	213	98	66
	300	3,850,000	375,000	238	177	215	124	86
37,500	400	2,350,000	385,000	216	177	211	113	75
	300	4,900,000	450,000	270	173	212	140	94
50,000	360	5,230,000	600,000	252	205	245	124	82

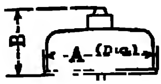
* Data by General Electric Company.

† Add 15% for shipping weight within the United States; 20% for foreign shipping weight.

TABLE 2

APPROXIMATE WEIGHTS AND DIMENSIONS VERTICAL WATER-WHEEL GENERATORS *

This table applies to 3-phase, 60-cycle, 0.8-pf generators without direct-connected exciters.



Rating, kva	Speed, rpm	Flywheel Effect, lb-ft²	Net Weight in Pounds		Dimensions in Inches		
			Generator	Exciter	A	B †	C
500	100	245,000	42,000	5,500	150	85	24
500	450	11,000	17,000	1,400	80	80	5
3,000	100	1,470,000	105,000	13,000	191	110	24
3,000	450	66,000	41,000	3,000	117	120	30
12,500	100	9,100,000	325,000	18,500	305	210	75
12,500	450	760,000	140,000	5,000	150	150	55
37,500	200	10,200,000	560,000	19,000	325	240	90
75,000	150	41,400,000	1,250,000	41,000	390	330	100
100,000	138.5	69,000,000	1,700,000	54,000	450	390	120
125,000	120	119,000,000	2,200,000	68,500	500	435	130

* Data by Westinghouse Electric Corporation.

† Including exciter.

weight of the heaviest piece, the rotor, which usually determines the capacity of the turbine-room crane, is approximately 50% of the total net weight of the generator and is more than 50% if the machine is designed for abnormal overspeed or flywheel effect.

4. Rating. The complete rating of a generator as given on its rating plate includes output, power factor, voltage, phase, frequency, speed, and current. The maximum output is limited by the allowable maximum temperature which the insulation will withstand safely. As it is difficult to measure the temperature of the hottest spot in the machine, a conventional "hottest-spot" allowance of 5 to 15 degrees Centigrade, depending upon method of measuring the temperature, is subtracted from the maximum allowable temperature to obtain the guaranteed observable temperature. The observable temperature is stated in centigrade degrees rise above an ambient or room temperature of 40° C and usually is specified for continuous operation, meaning operation continued long enough for the machine to attain its maximum temperature. Standardized temperatures for various kinds of insulation are given in the Standards of the American Institute of Electrical Engineers.

The *kilowatt output* of a hydro generator is chosen to match the output of the turbine, which depends upon the hydraulic flow and head at the site.

The turbine usually is rated on the basis of normal head and has a capacity some 10 to 20% greater at maximum head. The American Institute of Electrical Engineers' rules require that a generator shall operate continuously at a temperature not to exceed 60° C rise above 40° C ambient if it is provided with Class A (fibrous) insulation and not to exceed 80° C rise above 40° C ambient if provided with Class B (mica) insulation. A conservative method of matching the generator rating to the turbine rating, generally used, is to provide Class B insulation and to establish the generator kilowatt output about equal to the turbine kilowatt output, less generator losses, at normal head. With normal design the generator then will have about 15% greater capacity available for higher heads without exceeding 80° C rise and will therefore operate for most of its life at conservative temperatures.

The *kva rating* is equal to the kilowatt output divided by the power factor. Since the heating of the machine is a function of its kva output, the rating of a generator usually is stated in kva. The power factor should be chosen to match the load requirements as closely as reasonable. Low power factor requires a strong field If a generator is built for low-power-factor operation, such as 0.7, and is required to operate at a high power factor, such as 0.95, it must operate with its normal field strength greatly reduced and is likely to be somewhat unstable. Many hydro generators have been designed for 0.8 power factor to match their poor power-factor loads, but for a decade or more, 0.9 and higher power-factor generators have been in the majority.

Voltage is determined by the arrangement and connection of the stator conductors, the speed of rotation, and the field strength. Voltage ratings are available up to 25,000 volts. Higher voltages require less copper but more insulation and iron, and each kva and rpm rating has a corresponding economical voltage rating. When choosing a generator voltage, consideration should be given to the effect on the generator, switchgear, transformers, and wiring (see Chapter 40, Section 3).

Voltage adjustment of a generator operating open-circuited is accomplished by adjustment of field strength, by means of either the generator field rheostat or the exciter field rheostat. If a generator is connected to a system, adjustment of field strength tends to change both voltage and power factor.

Speed affects inversely the size, weight, and cost of the generator and, to some extent, of the turbine also. The speed of the unit is chosen, therefore, as the highest 60-cycle speed (see Chapter 40, Section 7) at which the turbine manufacturer will guarantee reasonable freedom from cavitation.

5. Efficiency. The true efficiency of a generator is the ratio of its useful output to its total input. The determination of the true efficiency involves the accurate measurement of the output and simultaneous input or the accurate measurement of all the losses. Accurate determination by either of these methods is impossible without the use of highly refined laboratory instruments and means of driving the machine at full load and absorbing its

output. Consequently a "conventional efficiency" which is readily obtained by measuring the principal losses with commercial instruments, and which is very close to the true efficiency, is universally used. The conventional efficiency of an alternator is defined in the Standards of the American Institute of Electrical Engineers and takes into account (1) core loss; (2) I^2R loss in armature, field, and field rheostat; (3) stray-load loss; (4) windage and bearing-friction losses; and (5) brush-friction and brush-contact losses. The efficiency depends upon the size, speed, and general design. Higher than average efficiencies can usually be obtained, at greater first cost, by careful design, and the expense is sometimes warranted by service conditions. Approximate efficiencies of typical standard alternating-current generators are given in Table 3.

TABLE 3

APPROXIMATE EFFICIENCIES OF STANDARD WATER-WHEEL GENERATORS, 3-PHASE, 60-CYCLE, 0.8 PF *

These efficiencies take into account the following losses: Core, armature I^2R, field I^2R, field rheostat I^2R, stray load, windage, bearing friction, brush friction, brush contact. Exciter losses are not included.

Rating, kva	Speed, rpm	Efficiency at Unity pf 50% Load	75% Load	100% Load	Efficiency at 0.8 pf 50% Load	75% Load	100% Load
500	100	91.5	91.8	91.9	88.0	89.6	89.7
500	450	92.2	93.6	94.0	90.1	91.8	92.5
3,000	100	94.4	95.2	95.3	93.0	93.9	94.0
3,000	450	94.9	95.7	96.0	93.2	94.1	94.7
12,500	100	95.7	96.4	96.5	94.6	95.4	95.7
12,500	450	95.6	96.5	96.9	94.6	95.7	96.2
37,500	200	96.4	97.0	97.2	95.6	96.3	96.6
75,000	150	96.5	97.1	97.3	95.6	96.3	96.6
100,000	138.5	96.7	97.2	97.4	95.8	96.4	96.6
125,000	120	96.7	97.2	97.4	95.8	96.4	96.6

* Data by Westinghouse Electric Corporation.

6. Regulation. The regulation of a generator is defined in the Standards of the American Institute of Electrical Engineers as the rise in voltage, expressed in percentage of normal rated-load voltage, that occurs when a specified load at specified power factor is reduced to zero. It is usual to specify the load as rated load and to give the regulation at unity and 0.8 power factors. A direct method of obtaining the regulation of a generator is by loading it to the specified load and power factor and then reducing the load to zero, measuring the voltage before and after removing the load, with speed and excitation adjusted to the same values for both measurements. This method is not always applicable to large generators on account of difficulty in obtaining the desired loads and power factors, and graphical meth-

ods are therefore used, as described in Section 7 and in the Standards of the A.I.E.E.

7. Characteristic Curves. The typical characteristic curves of an a-c generator are shown in Fig. 3. The curves usually obtained by test or calculated in preliminary design are (1) no-load saturation, (2) air gap, (3) short-circuit synchronous impedance, and (4) zero power factor. The no-load saturation curve shows the relation between field excitation and arma-

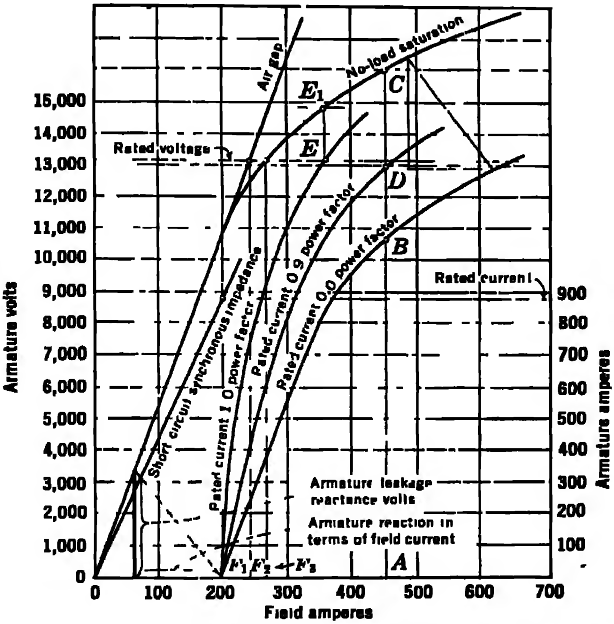


FIG. 3. A-c generator characteristic curves, 20,000-kva, 13,200-volt, 875-amp, 0.9 power factor, 3-phase, 60-cycle.

ture voltage at rated speed with armature open-circuited. The straight lower part indicates that with low values of excitation the voltage is proportional to the excitation. The curved upper portion indicates that as the excitation is increased above a certain point the iron becomes saturated and the voltage is no longer proportional to the excitation. The air-gap curve is merely the extension of the straight part of the saturation curve and does not take account of the saturation of the iron. The short-circuit synchronous-impedance curve shows the relation between field excitation and armature current at rated speed with armature short-circuited at its terminals. The curve is usually a straight line because the demagnetizing effect of the heavy armature current opposes the magnetizing effect of the field excitation and as a result there is little or no saturation. This demagnetizing effect always exists when the armature carries current and is known as armature reaction.

The zero-power-factor curve shows the relation between field excitation and armature voltage at rated speed with armature carrying rated current at zero power factor. The characteristic is sometimes obtained by loading the machine with an underexcited, unloaded, synchronous motor. The characteristic curves for various power factors may be obtained graphically as shown in Fig. 4. The method ignores resistance, which is usually negligible with respect to reactance. To obtain the 0.9-power-factor curve, draw lines *oa* and *ob* perpendicular to each other, and *oc* making with *ob* an angle, θ, whose cosine is 0.9. On the characteristic-curve sheet, select any field current, as *A*, and read voltages *AC* and *BC*. Lay off on line *oc*, Fig. 4, voltage $od = BC$, and with *d* as a center and *AC* as a radius cut *oa* at *e*. Returning to the curve sheet, locate point *D* by making $AD = oe$. Repeat for other values of field current until a sufficient number of points on the curve are obtained. Curves for other power factors may be obtained by a like process, making θ equal to $\cos^{-1}$ power factor. For unity power factor, line *oc* will coincide with *ob*. It will be seen that for rated current and voltage at 0.9 power factor a field current of 465 amperes is required.

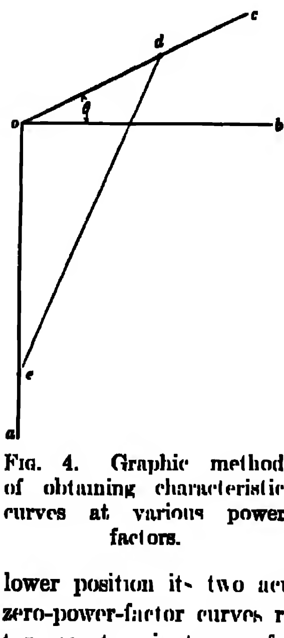


FIG. 4. Graphic method of obtaining characteristic curves at various power factors.

The armature reaction in terms of field current and the armature leakage reactance volts may be obtained graphically, referring to Fig. 3, by finding by trial and right-angle triangle of such shape that when moved parallel to its lower position its two acute corners will trace the no-load-saturation and zero-power-factor curves respectively. For the generator shown, the armature reaction in terms of field current is 135 amperes and the leakage reactance is 3400 volts, or 25.8%.

The synchronous reactance is defined as the voltage drop, in percentage of rated voltage, caused by the inherent armature reactance and the armature reaction at rated current. Referring to Fig. 3, the synchronous reactance is, according to some authorities, the ratio of field currents F_1/F_2, and according to others the ratio F_1/F_3. In most machines F_3 is only 5 to 15% greater than F_2 and, therefore, the discrepancy is small. Figure 3 shows the synchronous reactance to be 82.7% by the first method and 75.2% by the second method.

The sustained short-circuit current is obtained from the relation:

$$I_s = I\frac{F_s}{F_1} \qquad [1]$$

where I_s = sustained symmetrical rms short-circuit current per terminal in amperes;

I = rated current per terminal in amperes;

F_s = field current during short circuit;

F_1 = field current corresponding to rated current from the short-circuit curve.

The sustained short-circuit current of the alternator shown, if short-circuited at full load 0.9 power factor, is 2040 amperes.

Transient reactance includes the leakage of the field and of the armature and determines the total initial short-circuit current. On the occurrence of a short circuit the armature reaction quickly reduces the flux, and the initial current decreases in accordance with the decrement curve until the sustained value is reached. The transient reactance of standard water-wheel-driven generators varies from 15% for high-speed designs to 30% for low-speed designs.

Regulation may be obtained from the characteristic curves by reading the voltage from the saturation curve at any particular value of field current at which the regulation is desired. From Fig. 3 the regulation at field current corresponding to full load unity power factor is:

$$\text{Reg.} = \frac{E_1 - E}{E} = \frac{14{,}850 - 13{,}200}{13{,}200} = 0.125. \qquad [2]$$

For further information on characteristics see Section 43, Refs. 6 and 11, and also Chapter 40, Section 17, Refs. 15 and 20.

8. Flywheel Effect. Flywheel effect of a hydro unit is expressed in terms of weight of rotating parts times the square of the radius of gyration, WR^2. Most of the flywheel effect of a hydro unit is in the generator rotor. If more is required than is inherent in a normal generator design it usually can be obtained by increasing the weight of the rotor rim, but in some cases flywheels are added. A reasonable increase in flywheel effect over that of normal design usually can be obtained at reasonable cost. If carried beyond a reasonable limit, however, it increases the size, weight, and cost of the generator; increases the reactances; decreases the efficiency; requires a larger building and heavier crane; and ultimately reaches a condition where the margin of return is not justified.

Amount of flywheel effect needed depends upon many conditions. For a single generator on a small system, the WR^2 must be sufficient to insure satisfactory speed regulation under conditions of sudden load change This involves the governor, and, in some cases, the penstock and surge tank. For a long penstock it is necessary to use relatively slow gate operation to avoid water hammer, and this requires large flywheel effect to maintain satisfactory speed regulation. For a generator operating in parallel with others on a large system, the WR^2 must be sufficient to avoid hunting and

to insure stability in operation under conditions of sudden load changes and faults. This involves the characteristics of the other generators, of the power system, and of the load. (See Section 43, Refs. 6 and 11, and also Chapter 40, Section 17, Refs. 15 and 20.)

9. Ventilation. The electrical losses of a generator all appear as heat, which must be carried off by direct radiation and by ventilation. The moderate-sized generator is partially inclosed and provided with small fan blades mounted on the rotor. The larger-capacity generator is usually totally inclosed and provided with fan blades and with definite intake and discharge air-duct connections. There are several systems of directing the cooling air through the machine, among them the radial, the axial, and the circumferential, but all essentially take air in at one or both ends, force it through ducts between windings and between sections of the core, and discharge it again at the periphery.

Ducts and dampers are often provided so that the air can be taken from out-of-doors or from the station and can be discharged out-of-doors or into the station, thereby providing a flexible means of maintaining comfortable station temperature during all seasons of the year. It is good practice to locate the air intake over the tailrace at a safe distance above extreme high water. The air so obtained is generally cool and clean and contains some moisture, which renders it much more efficient as a cooling medium. When the fan action of the rotor itself is depended on to circulate the air, the ducts should be short and direct and should have well-rounded corners. It is advisable to submit the duct design to the generator manufacturer before the final design of the rotor fans is completed. If the ducts are of excessively high resistance an external fan may be required. Roughly, 100 cu ft of air per minute will be required for each kilowatt of loss in the generator, and the velocity in the ducts should not exceed 1000 to 1500 ft per minute. Exact data on the particular machine in question should be obtained from the manufacturer.

Recirculation of air within the housing and cooling by means of water-cooled heat exchangers are very common in modern generators. The windings are kept much cleaner than when open ventilation is used even at favorable sites where outdoor air is comparatively clean. The inclosing housing also permits the effective use of carbon dioxide as a fire-extinguishing agent.

10. Temperature Detectors. In order to determine the temperature of the interior of the generator armature while operating under load, temperature detectors are placed between adjacent coils or between coils and core, the leads of the detectors being brought out and connected to indicating instruments on the switchboard. Usually, at least ten detectors are installed in a generator and all leads brought out to a terminal block on the frame. It is usually sufficient to connect five to the switchboard and to reserve the other five for use in case any should become accidentally injured. The terminals on the generator frame should be provided with a protective device to prevent any abnormal voltage from the generator armature being carried to the

switchboard. Two general types of detectors are in common use: the resistance coil, and the thermocouple.

The resistance coil consists of a flat coil of copper or alloy wire having a resistance, usually of 10 ohms, and a constant temperature coefficient of resistance. The instrument operates on the differential D'Arsonval principle. Three leads are used between the resistance coil and the instrument so as to eliminate the effect of the resistance of the leads.

The thermocouple consists of two special dissimilar metals, such as Constantan and copper, welded together to form an electrolytic couple. The two leads, one of Constantan, the other of copper, are carried to the instrument and there joined together to form the cold joint. The instrument consists of a sensitive galvanometer and depends for its operation on the potential generated by virtue of the difference in temperature between the two junctions of the dissimilar metals.

11. Fire Protection. Most hydro generators are provided with fire protection to reduce the damage resulting from internal faults. Water has been used to a considerable extent. It is sprayed onto the stator windings by means of rings of perforated pipe behind the end turns. To positively prevent application of water by leakage or by accident, two valves sometimes are used in series in the supply line with a small drip between valves, or a short section of flexible pipe is sometimes provided which normally is disconnected.

Carbon dioxide currently is used more extensively than any other means of fire protection. It both quenches the flame and cools the heated parts so as to prevent re-igniting. Special nozzles and carefully proportioned piping allow the liquid carbon dioxide from banks of high-pressure cylinders to be delivered into the generator without freezing. The discharge usually is initiated automatically by thermostats or by differential relay action. The initial dis-

Number of Generators Served by CO_2 System	Data on Each Generator			Number of 50-lb CO_2 Cylinders in	
	Kva	Rpm	Air Volume of Housing, cu ft	Initial Bank	Delayed Bank
1	30,000	200	4,500	7	6
2	40,000	225	5,650	10	6
2	56,000	112.5	10,700	20	20
2	64,000	120	10,200	18	8
3	75,000	150	9,800	14	10
4	27,000	69.2	23,000	38	30
4	28,000	100	11,000	18	10
4	30,000	75	14,000	24	12
4	33,333	94.7	14,000	20	14
4	35,555	105.8	9,300	14	10
5	33,333	94.7	10,500	16	10
5	35,000	78.3	22,000	30	20
6	40,000	81.8	16,000	20	20

charge is proportioned to produce a concentration of about 50% carbon dioxide, which is sufficient to quench a fire; delayed discharges are timed to maintain the concentration at not less than 25% to prevent re-ignition. The generator housing obviously should be fairly airtight. A single installation in a station will serve several generators through automatically controlled routing valves. Essential data on a number of recent installations are as shown in table on p. 957.

12. Installation. Small generators can be completely assembled in the generator factory and often are installed by the powerhouse builder. Medium and large generators are sufficiently assembled in the factory to assure the various fits and then are partially disassembled and shipped in parts. Field erection then becomes a major operation. With very large generators the field and armature cores are sometimes stacked and the windings installed in the field. It is usual practice then to make the field erection a part of the purchase contract, but the powerhouse builder often sets foundation bolts and sole plates in accordance with the manufacturer's instructions.

13. Drying Out. Generators that are assembled and tested before being shipped are usually dry and ready to run when they leave the factory. A machine of this type, however, as well as the coils of a machine that is assembled on its foundation, is likely to absorb moisture during transit, storage, and erection. Any such absorbed moisture is likely to cause an insulation failure and should be carefully driven out before the machine is brought up to voltage. A recommended method of drying is to run the machine at normal speed with the armature short-circuited beyond the ammeters so as to circulate enough current to warm the windings. The drying out is usually a part of the erection by the manufacturer, and if it is done by anyone else the manufacturer's instructions should be carefully followed. For generators not having imbedded temperature detectors, it is usually recommended that the armature current, which is adjusted by field control, be maintained at about 110% of normal. For generators with temperature coils, it is recommended that the temperature be adjusted to about 75° to 80° C (total temperature, not rise). In either case, it is advisable to check the temperature by thermometers placed about the stator.

The drying out should be continued until insulation resistance measurements taken about every 4 to 8 hours indicate that the windings are dry. The trend of the resistance values during dryout are much more indicative than the individual readings. During the course of the dryout, the resistance decreases rapidly for the first few hours while the temperature rises. After the temperature becomes steady, the resistance rises rapidly at first, then gradually becomes fairly steady within 3 to 6 days, depending upon the size of the machine. It usually is assumed that the winding is dry after its resistance has been fairly steady for 24 hours. A dry, clean stator winding should have an insulation resistance in megohms at 75° C of not less than:

$$\frac{\text{Rated voltage of machine in volts}}{(\text{Rated kva} \times 0.01) + 1000}$$

Since the rotor usually dries out faster than the stator, its resistance trend ordinarily is not observed. A dry, clean field winding for 125- or 250-volt excitation should have a resistance at 75° C of not less than ½ to 1 megohm [8, 10].

14. Measuring Insulation Resistance. The insulation resistance may be measured by a direct-reading ohmmeter, by a resistance bridge, or by a high-resistance voltmeter and d-c source. The connections for the voltmeter method are shown in Fig. 5. Two readings are required, E_1 with only

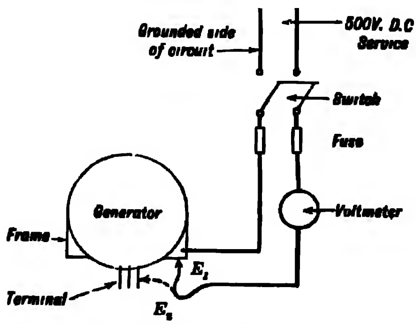


FIG. 5. Diagram of connections for measuring insulation resistance by voltmeter method.

the voltmeter in circuit, and E_2 with the voltmeter and the insulation in circuit. The insulation resistance is found from the following equation:

$$R_x = \frac{R_v(E_1 - E_2)}{E_2} \qquad [3]$$

where R_x = resistance of insulation in ohms;
R_v = resistance of voltmeter in ohms;
E_1 = voltmeter reading with voltmeter only in circuit;
E_2 = voltmeter reading with voltmeter and insulation in circuit.

In making the test the following cautions should be observed:

1. If either side of the d-c circuit is grounded, the grounded side must be connected directly to the generator frame.
2. The voltmeter must be connected in the ungrounded wire close to the source of supply.
3. The wire between the voltmeter and the generator terminal must be very carefully supported, clear of all grounded objects, to prevent any possible leakage through its own insulation. It is advisable to suspend it free in the air.
4. The voltmeter must have a high resistance, preferably about equal to the resistance being measured.

15. Dielectric Test. Immediately after the stator is considered dry, it is usual to apply the specified dielectric test between the winding and the frame before the temperature falls below 75° C. The American Institute of Elec-

trical Engineers' standard test for a generator stator is by alternating voltage whose effective value is twice the generator rated line-to-line voltage plus 1000 volts (28,600 volts for a 13,800-volt generator). The field test is by alternating voltage whose effective value is 10 times the excitation voltage but not less than 1500 volts (2500 volts for a 250-volt field). Some users specify twice this test voltage for fields of large generators. The exciter test is the same as the field test, but it is not usual to double the standard test as is sometimes done for the field. For both stator and field, the duration of the test voltage is 1 minute. In case of failure under test, the value and duration of retest after repair usually are agreed upon by manufacturer and purchaser, the voltage and duration of test obtained before failure occurred being taken into account.

16. Starting. Before starting a new alternator for the first time, it is advisable to give it a very thorough inspection to see that there are no loose pieces of iron or tools lying near that might be drawn into the rotating parts by the magnetic pull, that all air gaps are clear, that all electrical clearances are ample, that bearings are well supplied with clean oil, that oil rings are free to turn, and that brushes and brush holders are in good order. The external circuits should also be carefully inspected and cleared for operation. The main and field breakers should be open. The machine should be started slowly by its prime mover and gradually brought up to speed. With the rheostat so adjusted that all its resistance is in the field circuit, the field breaker should be closed and the resistance then gradually cut out until the machine voltage comes up to normal. The main breaker may then be closed, connecting the alternator to the bus if the bus is not already alive from any other source. In the majority of cases the alternator is not intended to operate on an isolated system but must operate in parallel with other alternators or on a system already in operation. In this case the new alternator must be properly phased out and synchronized before being connected to the bus (see Section 17).

17. Phasing Out and Synchronizing. *Phasing out* is the term applied to the operations which precede the first connection of a newly installed generator to the bus. These operations are for the purpose of:

1. Checking the correctness of the wiring, including main generator leads, potential transformer connections, and synchroscope wiring.
2. Checking the phase rotation of the new generator to determine whether it agrees with that of the bus.

The wiring can be checked readily by disconnecting the generator leads at the generator terminals and, with the wiring terminals properly insulated, closing the generator synchronizing switch and circuit-breaker. This will connect the generator potential transformers to the bus in the same manner as they will be when the unit is in operation, and obviously the voltage applied to them will be correct both as to phase rotation and phase position. If a single-pointer synchroscope is used, it will indicate in one of six positions

which may be designated by clock hour-hand positions. If the scope shows 4 or 8 o'clock, the synchroscope wiring is connected to the wrong phase of the generator leads. The potential transformer used for synchronizing must have its primary leads disconnected from the phase to which they are now connected and moved to the correct phase. If the scope indicates 6 o'clock, the two secondary leads to the synchroscope are reversed. If it indicates 2 or 10 o'clock, it is both connected to the wrong phase and has the secondary leads reversed. If it indicates 12 o'clock, the connections are correct.

The phase rotation can be checked readily by means of a phase rotation meter or small 3-phase induction motor in conjunction with two potential or distribution transformers. With the generator circuit-breaker open, the transformer primaries are connected to the open ends of the generator leads from the breaker (where opened for the wiring check as described above), and the secondaries are connected to the phase rotation meter or motor. The breaker is then closed. The direction of rotation of meter or motor should be noted and the breaker reopened. Without disturbing the secondary wiring, the primary leads of the transformers are transferred lead-by-lead from the breaker leads to the corresponding generator terminals, taking care not to cross phases. Field is then applied to the generator and the unit started. If the phase rotation meter or motor rotates in the same direction as before, the phase rotation is correct; if in the opposite direction, a pair of generator main leads must be reversed either between the gap and the generator or between the gap and the main bus. In the latter case, however, care must be taken that this change does not affect the synchroscope wiring and as an extra precaution the wiring check should be repeated.

When both the preceding conditions are ratified the generator may be connected permanently to its leads, care being taken that the circuit-breaker is open.

Synchronizing is the regular operation followed each time the unit is put into service. Before the circuit-breaker is closed, three conditions must be satisfied to avoid voltage dips and possible interruption of service. These are:

1. The voltage of the generator must be the same as that of the bus.
2 The frequency of the generator must be the same as that of the bus.
3. The phase position of the generator must be the same as that of the bus.

The first condition is determined by two voltmeters or one voltmeter and a switch to transfer the meter from the bus potential transformers to the generator potential transformers. The second and third are determined by the synchroscope. Counterclockwise rotation indicates low frequency of the generator; clockwise rotation, high frequency of the generator. When the synchroscope pointer is stationary, the frequency of the generator is the same as that of the bus, and when the pointer is at 12 o'clock the phase position is the same. This is the correct position for closing the circuit-breaker connecting the generator to the bus. A small difference in any or all of the three preceding requirements is permissible without undue fluctuation of voltage.

The larger the difference, the greater will be the voltage disturbance, and if sufficient difference exists the voltage will be lowered to the point where there is insufficient synchronizing power to pull the generator into step. Closing the circuit-breaker with the generator out of synchronism with the system is in effect applying a short circuit to the system. Good operators close the circuit-breaker control switch slightly before the pointer reaches 12 o'clock in order to allow for the time necessary to close the breaker. The amount of advance is dependent upon the speed at which the pointer is moving and the speed of the circuit-breaker in closing.

18. Parallel Operation. Generating units that are to operate in parallel on the same bus and share the kilowatt load roughly in proportion to their ratings must have prime movers with speed-droop characteristics so that an increase in load will tend to decrease the speed, and vice versa. Governors for hydro turbines usually have speed-droop controls by which the amount of speed droop at rated load may be adjusted from zero to 6% of rated speed. At any adjustment, the speed droop at other loads is roughly proportional to the load.

Alternating-current generators operating in parallel also must share the power factor or reactive kva (kilovars). This is accomplished by means of the voltage regulator acting on a field rheostat which, as mentioned in Section 4, tends to control both voltage and power factor. By the addition of cross-current compensation the voltage regulator is given a voltage-droop characteristic which can be adjusted to equalize the power factor on the individual generators while the speed-droop characteristic of the governor can be adjusted to equalize the load.

19. Load-frequency Control. On an important power system it is necessary to maintain practically constant speed regardless of load, and, since many synchronous clocks are in operation all over the system, it also is necessary to maintain correct integrated frequency or time. Therefore, load-frequency-control equipment which automatically compares the frequency and time with a highly accurate pendulum or tuning fork and adjusts the frequency as needed is usually installed in the generating stations of a large power system. The pendulum or tuning fork usually is checked daily or oftener against astronomical time signals. Since frequency and time can be set and maintained at only one point on an interconnected system, a single station at a time is assigned this duty. The others accept this standard and control only their generation to meet load and tie-line requirements.

20. Alternating-current Generator Specification. The following brief outline is intended as a guide to the principal items to be included in a specification for an important hydro generator:

General. Number wanted; scope of work by purchaser and by contractor; available transporting, handling, and erecting facilities; standards of workmanship, test, and performance, such as A.I.E.E., A.S.A., N.E.M.A., A.S.T.M., N.E.S.C.; shop inspection, assembly, testing, painting, match marking, packing;

field tests; preliminary operation; acceptance tests. (See A.S.A. Standard C50 and A.I.E.E. Standard 503.)

Type and Rating. Position of shaft; number and location of bearings; type of cooling; type of excitation and exciter drive; continuous output kva and temperature rise; power factor; voltage; amperes; speed; phases; cycles; excitation voltage.

Electrical Requirements. Continuous outputs when generating at rated power factor, when operating as a synchronous condenser overexcited, and when charging a transmission line underexcited, without exceeding A.I.E.E. temperature limitations; class of insulation in armature and field; short-circuit ratio; connections of armature windings; insulation of neutral ends of windings if wye-connected; telephone interference factors; wave-form-deviation factor; nominal exciter response ratio; exciter ceiling voltage; exciter insulation class; exciter temperature rise; exciter field connections; pilot exciter requirements; type of voltage regulator; voltage regulator sensitivity and time of response.

Mechanical Requirements and Data. Flywheel effect; runaway speed; rotation direction; thrust bearing load in addition to weight of generator rotating parts; cooling water temperature; limiting over-all diameter of housing; internal diameter of stator to pass turbine parts; turbine normal output, maximum output, head-cover diameter, pit diameter; elevation of shaft coupling in relation to stator sole plates.

Construction. Stator frame, sole plates, foundation bolts, core, windings, terminals; rotor hub, spider, core, poles, windings, slip rings, damper windings; shaft material, internal bore, flange for coupling to turbine shaft, coupling bolts and guard, flange, or other connection to rotor and to bearings; assembly of generator and turbine shafts and checks for accuracy; bearing type, lubrication, performance at low speeds and at runaway speeds; bearing brackets, sole plates, foundation bolts; brakes, braking surfaces, stopping time, station air pressure, jacks, piping; cooling system for generator and exciters, heat exchangers, piping, valves, vents, fans on rotor, air baffles; housing shape, finish, access doors, airtightness, internal lighting, walkways, stairs; internal piping, wiring, internal connections from main exciter brushes to generator slip-ring brushes, connections to station piping and wiring.

Excitation Equipment. Main exciter rheostat with shunting contactors and relays for high-speed excitation; exciter field breaker with discharge resistor; pilot exciter rheostat; pilot exciter field-reducing relay; panels, wiring, housing.

Neutral Equipment. Breaker; disconnect; reactor rating, temperature rise, time of operation, insulation; panel, wiring, housing.

Accessories. Temperature detectors in stator windings, in bearing shoes, in oil reservoirs, in air ducts; indicating thermometers with alarm contacts for bearing shoes; oil-level gages; oil-level remote indicators with alarm contacts; shunts for field current; shunts and special slip-ring brushes for field temperature recorder; current transformers within the neutral connection and matching transformers for the other end of the differential relay circuit; insulation against stray shaft currents; space heaters for stator; provision for governor generator and overspeed switch; provision for Kaplan oil head; voltage regulator; arresters and capacitors for surge protection; portable oil pump for jacking rotor; wrenches, wrenchboard, tools, jack for removing coupling bolts; eye bolts, slings, collars, keys used for assembly; erecting pedestal for rotor assembly; lowering device for bearings or other parts; spare parts.

Guaranteed Performance Data by Bidder. Over-all efficiency at rated voltage and speed, at 50, 75, 100, and 115% of rated kva, and at unity and 0.9 lagging power factors; maximum safe indicated temperature of stator windings; maximum safe indicated temperature of rotor windings; ratings of exciters.

Design Data by Bidder. Type designation; exceptions to specification; bearing dimensions, load data, areas, construction, adjustments, accessibility; weights of stator section, shaft, rotor, heaviest crane lift, complete generator; description and performance of damper windings; method of assembling and disassembling generator and provision for removing turbine parts; method of attaching rotor to shaft; list of special tools required; material stresses and safety factors for all rotating parts; quantity of cooling water for generator air and bearing oil; quantity of oil; net air volume in housing; descriptions of voltage regulator, neutral reactor, neutral breaker; synchronous, transient, subtransient, negative sequence, zero sequence reactances.

Data by Contractor. Losses at various loads and at unity and rated power factors; short-circuit currents; resistances, reactances, time constants; field currents; slip-ring voltages; voltage regulation; continuous capacity when condensing and when charging a line; flywheel effect; pressure drop through coolers; net air volume of housing, weights of parts, total weight.

Shop Tests. Each generator armature and field coil dielectric; each exciter dielectric; one exciter temperature rise, saturation, regulation, tooth ripple, armature resistance, field resistance, losses, efficiency, overload commutation, response ratio, ceiling voltage; exciter characteristic curves of efficiency, armature copper loss, field copper loss, core and stray losses, total loss, saturation open-circuit volts, regulation volts; individual tests of regulators, breakers, reactors, rheostats, current transformers, gages, instruments.

Field Tests and Preliminary Operation. Shaft alinement, dryout, insulation resistance, dielectric tests, preliminary operation, final adjustments before commercial operation.

Acceptance Tests on Each Generator. No-load saturation; short-circuit saturation; telephone interference factors, no-load balanced and no-load residual; resistance of armature and field; operation of pilot exciter field-reducing relay; main exciter rheostat time for full travel; voltage-regulator operation; sensitivity and response.

Acceptance Tests on One Generator. Efficiency by losses (friction and windage losses of entire unit to be divided between generator and turbine in proportion to estimates previously mutually agreed upon by the generator and turbine manufacturers); temperature rise at rated output, cooling-water quantities and temperatures; wave form by oscillogram of each phase on open circuit; short-circuit ratio; synchronous reactance; flywheel effect; braking time from half speed to zero speed with torque reduced to 1% of rated and with excitation removed.

21. Load Tests. It is sometimes necessary to provide an artificial load for testing a newly installed generator or water wheel, because connections to the prospective system load are not completed or system-load conditions are not suitable for tests. Various forms of energy-dissipating rheostats have been devised for this purpose (see Section 22). The rheostat forms a suitable load as far as the water wheel is concerned, but, owing to its nonadjustable unity power factor it is not so satisfactory for testing a generator. Where a second generator is available a simple scheme consists of connecting the two generators together with one phase reversed, so that the second generator will be driven as a motor in reverse direction against the resistance of its water wheel. Various loads and power factors can be obtained by adjusting the gate opening and field rheostat of the motoring unit [1].

22. Energy-dissipating Rheostats. A rheostat used as a load in testing water wheels and generators must be capable of dissipating large amounts

of energy. Rough adjustment of the load throughout the range of the water-wheel capacity is a necessary feature of the rheostat, but fine adjustment may be made by adjusting the generator voltage. Constancy of load at each step to within 1 or 2% for several minutes is essential to allow proper adjustment of the water wheel and governor and the recording of the various readings.

For voltages up to 500 volts, load rheostats may be made of coils of resistance wire immersed in water. The load is determined by the material and proportions of the coil, and the heat is absorbed by the water. Adjustment is made by taps or by interconnections. For voltages in this range, iron pipes also have been used as load rheostats, connected in suitable lengths and cooled by passing water through them.

For voltages up to 1000 volts, electrolytic rheostats, consisting of plate or rod electrodes dipped into an electrolytic solution, have been used successfully. Electrodes may be of carbon, copper, lead, or iron, and electrolyte of acid, alkali, or salt content, but care must be taken to avoid rapid chemical decomposition. Adjustment of load may be obtained by varying the spacing of electrodes, depth of immersion, or strength of electrolyte.

For voltages higher than 1000 volts, the so-called water rheostat, consisting of electrodes dipped directly into the headwater or tailwater, has been used almost universally. Being essentially an electrolytic rheostat, using the natural river water as an electrolyte, it differs in design and characteristics over an enormous range, depending upon the resistance of the water at the site. Load adjustment is obtained by varying either the spacing of electrodes or their depth of immersion. For high voltage, high-resistance water is necessary. Where the water is brackish or for other reasons has low resistance, a rheostat of this type may be entirely unsatisfactory. The flow of water about the electrodes must be sufficient to carry away the heat but not so turbulent as to affect the constancy of the load. Usually the tailrace is satisfactory if the rheostat is located away from the direct discharge from the wheels. The electrodes may be of wrought iron or steel. Their active area should be large enough to prevent boiling of the water. Rheostats of this type have been used with voltages as high as 30 or 40 kv.

The following data taken from successful installations of water rheostats indicate typical designs and the wide range of proportions necessitated by different values of water resistance. Depth of immersion as used herein refers to the depth of the wetted portion of the electrode, that is, distance from bottom of electrode to surface of water.

1 Three electrodes of solid iron rod, each ¾ in. in diameter, 24 in. long, wrapped with rubber for a distance of 10 in. above and 10 in. below the water surface, lower end 4 in. long exposed to water. Electrodes spaced on 41-ft triangle. Load 5500 kw at 13,200 volts, 3-phase, 60-cycle. Electrodes lasted 5 hr. Water was of low resistance owing to contamination by a number of factories.

2. Three electrodes of steel plate, each ¼ in. thick, 77 in. wide, 125 in. long, bent to an angle of 120° and assembled 15 in. apart to form a Y, immersed completely. Load 16,000 kw at 80° F and 12,000 kw at 34° F at 14,300 volts, 3-phase, 60-cycle. Load very steady. Electrodes had long life.

3. Six electrodes of 6-in. iron pipe, 10 ft long, spaced on two 3-ft triangles, centers of triangles 15 ft apart, triangles connected in parallel, immersed 9 ft 6 in. deep in forebay. Load 890 kw, 4000 volts, 3-phase, 60-cycle, steady within 0.2% for several hours.

4. Two sheet-iron plates each 39⅝ in. square, spaced 94.5 in. on centers, in tailrace. Loads single-phase, 25-cycle, as follows:

Depth of Immersion, in.	Volts	Amperes	Kw
6.3	24,500	27.9	665
8.9	28,000	37.0	1,020
11.7	29,550	42.5	1,250
9.0	30,950	41.2	1,280

5. Three electrodes of 6-in. wrought-iron pipe, each 8 ft 10 in long with flange on lower end, spaced on 12-ft triangle, in tailrace. Water resistance 3844 ohms per cm cube at 24° C. No perceptible heating or destruction of electrodes. Loads at 13,800 volts, 3-phase, 60-cycle, as follows:

Depth of Immersion, in.	Kw
3	2,500
13	10,000
33	15,000
53	20,000
73	25,000
93	30,000

6. Three electrodes of 4-in. wrought-iron pipe, 8 ft. long, hung at corners of a 3-ft triangle. With 6 ft of electrodes immersed in river water, load was 2500 kw at 11,000 volts. Fractional loads were fairly proportional to depth of immersion.

7. Three electrodes of 16-in. cast-iron flanged pipe, 9 ft long, hung at corners of a 4-ft triangle in an 8-ft by 8-ft wood box immersed in river water. With 7 ft of electrodes immersed, about 1200 kw at 2300 volts was obtained. Small amounts of saturated salt solution poured slowly into the box occasionally in conjunction with field rheostat adjustment held the load at about 3125 kw.

8. Three electrodes of 1-in. iron pipe, at corners of a 6-ft triangle, immersed 3 ft in river water gave about 3400 kw at 33,000 volts. Loads up to 5000 kw and as low as 100 kw were obtained by nearly proportional depths of immersion.

For description of rheostats used at Hoover Dam and Bonneville Dam, see Section 43, Refs. 3 and 4.

23. Generator Charging Transmission Line. When a generator is connected to one end of a long transmission line which is open-circuited at the other end, the susceptance of the transmission line causes a low-power-factor leading current to flow through the generator windings. The armature reaction within the generator, as a result of this leading current, strengthens the generator field flux and raises the voltage. This magnetizing action of an unloaded transmission line is often so great that a single generator connected to it may have its voltage built up to a dangerous value, even with very little or no excitation applied. It is, therefore, sometimes necessary to provide means of reversing the excitation to offset the magnetizing action

or to avoid connecting a single generator to an unloaded line. If the situation is foreseen, the design of the generator can sometimes be modified so as to avoid excessive voltage from this cause [2].

24. Excitation. The excitation of a generator is the power input required by the field winding to maintain the necessary intensity of magnetic flux. The effective flux is the resultant of the flux produced by the field current and that produced by the armature current. The latter is called armature reaction. With lagging power factor, the armature reaction opposes the field flux; with leading power factor, the armature reaction assists the field flux. Therefore, much greater field current is required with lagging power factor and heavy load than with leading power factor and light load. Since the power developed by the generator is proportional to the product of the magnetic field strength and the speed with which the field flux cuts the armature conductors, it is evident that the slower the speed of rotation the greater must be the excitation for a given power rating. Approximate excitation requirements of standard water-wheel-driven generators are given in Table 4.

TABLE 4

APPROXIMATE EXCITATION REQUIREMENTS OF WATER-WHEEL-DRIVEN GENERATORS *

Rating, kva	Speed, rpm	Excitation Requirements in Kilowatts: At Unity pf	At 0.8-pf Lag
100	1,200	2	3
	360	4	5
1 000	600	9	12
	150	15	20
5,000	600	25	35
	150	45	60
10,000	400	45	60
	150	65	85
20,000	360	75	100
	120	105	140
30,000	360	95	125
	120	135	180

* Data from General Electric Company.

Quick response excitation, that is, an excitation system which quickly responds to sudden change in generator voltage by immediately supplying the necessary increase or decrease of field current as required, has proved so valuable in increasing the stability of a power system that it is almost universal practice to provide quick response excitation for water-wheel-driven generators. It is accomplished principally by designing the exciter with a high

"ceiling voltage," that is, a high upper limit to which the exciter voltage will rise with full field, providing a pilot exciter, which is a small exciter to supply constant voltage to the exciter fields, and providing a quick-acting voltage regulator.

25. Excitation Systems. Performance and reliability are the prime requirements of an excitation system. Other important considerations are low first cost, economy of operation, simplicity, and convenience. The exciters should be of good design and liberal size; the method of drive should be reliable; the wiring should be short, simple, and carefully installed; the method of control should be convenient and simple; and reserve capacity should be provided.

The centralized excitation system, involving a bus fed by one or more exciters and feeding all the generator fields, has the advantage that the minimum number of large exciters may be used. A further possible advantage is that a battery may be floated on the bus as an emergency source. The principal objection to the centralized system is the possibility of a ground or other disturbance in one part affecting the entire system. The ideal centralized system would have three identical exciters, one of which could always be held in reserve.

The individual excitation system, involving an individual exciter associated with each generator, has the advantage that each generator with its exciter constitutes an independent unit not likely to be affected by faults originating in other units. A further advantage is that the exciter connections can usually be made shorter and simpler. A possible objection to the individual system, for a station having a large number of small generators, is that the exciters are comparatively small and inefficient.

Motor drive is convenient, inexpensive, and fairly efficient. Induction motors are most simple, convenient, and reliable, although synchronous motors may be used if they are specially designed for stable operation during system disturbances. The source of energy to drive the motors must be uniform and reliable.

Water-wheel drive is relatively unreliable, expensive, and inefficient for small individual exciters, but it is quite feasible for the larger exciters of a centralized system. At least one exciter must be driven by a prime mover for purposes of starting the station.

Dual drive, that is, by means of a motor on one end and a water wheel on the other, occasionally is advantageous. The water wheel can be used when starting the station, and either the water wheel or the motor for normal operation. Both the motor and the water wheel may be connected to their respective sources of energy, and the governor so adjusted that practically all the energy is taken from one source or the other as desired. Then, in case of failure of the driving source, the standby source automatically will assume the load.

Direct drive from the main unit is very efficient, reliable, convenient, and, except for very slow-speed units, inexpensive. It is by far the most com-

monly used of all drives. Possible objections are that the generator voltage is particularly sensitive to speed fluctuations and that trouble with an exciter may cause the retirement of a large main unit. The first objection is nullified partially by the use of a pilot exciter.

A storage battery provides a very reliable immediate standby source. Its high first cost, poor efficiency, and large space requirements, however, practically limit its usefulness to merely a momentary standby for use only while bringing a reserve exciter into service or to support the pilot exciter. For this reason, its greatest usefulness is obtained by floating it across the exciter bus, so that in case of failure of the exciter the generator excitation will not be interrupted, or at least by providing automatic transfer switches so that the time of interruption may be reduced to the minimum. An equally reliable standby can be obtained by operating a spare exciter continuously. Such an immediate standby, however, is seldom needed except in stations feeding out directly to important commercial customers, and batteries or continuously operating spare exciters are seldom used in hydroelectric stations.

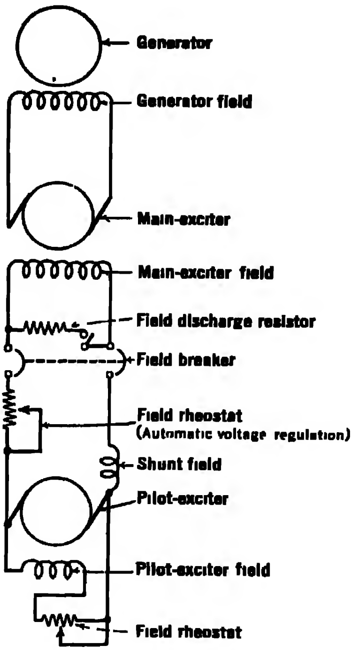


FIG. 6. Elementary excitation circuit.

The voltage of the excitation system is generally 125 volts for small stations and 250 volts for large stations. For very large stations, higher voltages have been suggested in order to reduce losses, cable sizes, and duty on slip rings.

26. Excitation-system Wiring. Wiring for the excitation system should be short, compact, of ample capacity, well supported, and carefully safeguarded. The insulation should be designed for eight to ten times the normal excitation voltage, to take care of transient pressures due to generator short circuits.

The elementary excitation circuit shown in Fig. 6 is typical of modern practice with large generators, each having its own main and pilot exciters. The generator field rheostat often used for small installations is omitted for larger generators as its space, cost, and losses make it uneconomical. Automatic voltage regulation is on the main exciter field. The pilot exciter provides a constant voltage for the main exciter field, resulting in faster voltage control and excitation response and in greater generator stability. The pilot exciter field is self-excited, and the manually operated pilot exciter field rheostat does not ordinarily require adjustment after being set at the proper value initially.

Breakers and rheostats are often hand-operated in small stations, but usually great advantage can be gained by making them electrically operated, since they can then be located most advantageously in the station to give short connections. Where exciters are bussed, the exciter main breaker is usually made automatic on reverse power, in order to disconnect the exciter in case of internal failure. The generator field breaker or the exciter field breaker usually is arranged to be tripped by generator relay action. It is not usually considered advisable to place overload protection on exciters as it is better to risk injury to the exciter than unnecessary operation of the automatic device. For instance, a sudden overload or short circuit in the generator circuit might cause sufficient rush of current in the exciter circuit to trip its breaker if it were automatic.

The excitation circuit must never be suddenly opened. The excessive voltage that would be built up by the sudden releasing of the energy stored in the inductive field winding might be sufficient to puncture the field insulation. To guard against this, the field breaker, which alone is used to break the circuit, is provided with discharge clips to be connected to the discharge resistor which is always furnished with the generator. As the breaker opens it connects the resistor across the field terminals and the energy is dissipated in the resistor. In some instances, interlocks are provided to prevent the opening of any breaker in the circuit until the field breaker is open, but to avoid complexity it is generally considered better to depend on the operator for proper sequence of operation.

27. Exciters. An exciter is a direct-current generator designed especially to supply excitation for synchronous machines. The principal aim in the design for this purpose is to obtain stability under a wide range of voltage. For use with an automatic voltage regulator, the exciter should be designed so that its voltage will respond quickly to a change in its field current For parallel operation with other exciters, it should have a drooping voltage-current characteristic at all operating voltages, unless such characteristic is provided by the voltage regulator and cross-current compensation. Interpole windings are now practically standard for improving commutation. An exciter designed for use with an automatic voltage regulator is not necessarily as stable at lower voltages as one designed for manual voltage regulation, and, therefore, it is important to specify for which method of operation the exciter should be designed.

Approximate weights and dimensions of standard exciters are given in Table 5.

The compound-wound exciter is usually preferred to the shunt-wound exciter where the alternating-current voltage regulation is accomplished manually, as the compound exciter requires less frequent adjustment to meet changing load conditions Where automatic voltage regulators are used there is little choice between the compound exciter and the shunt exciter as far as the action of the regulator is concerned. The compound exciter is more stable

TABLE 5 *

APPROXIMATE WEIGHTS AND DIMENSIONS OF EXCITERS

Type	Rating, kw	Speed, rpm	Net Weight,† lb	Dimensions, in. Length	Width	Height
Coupled (horizontal)	25	150	4,100	32	44	42
	50	150	7,300	42	56	54
	100	150	12,000	53	71	68
	150	120	17,000	57	79	76
	200	120	20,000	57	85	84
Belted (including pulley and base)	25	700	1,990	57	30	35
	50	700	3,170	70	33	39
	100	700	5,150	85	39	46
	150	700	6,950	92	42	48
Motor-driven (including motor)	25	1,800	1,785	63	22	27
	50	1,200	3,900	77	30	37
	100	1,200	6,200	94	42	42
	150	1,200	9,000	109	42	42
	200	1,200	9,900	111	42	42
	250	1,200	11,600	121	42	42
	300	1 200	13,500	119	50	53

* Data from General Electric Company.

† Shipping weights are 12 to 15% greater than net weights.

in its operation during system disturbances than the shunt exciter. On the other hand, there are cases where, because of the magnetizing effect on the generator of a long transmission line, it is necessary to be able to reduce the exciter voltage to a very low value, and then the shunt exciter possesses some advantages. Compound exciters, if operated in parallel, require equalizer connections which complicate the station wiring.

28. Field Rheostats. For small generators, in order to allow a reasonable margin of safety for variations in design and materials, it is customary to wind the generator field so that about 90% of the exciter voltage will be required for rated load and power factor. A rheostat is then provided in the generator field circuit to control the excitation as required. The rheostat is usually designed with sufficient resistance to reduce the generator voltage to about 85% of normal with no load, full speed, generator cold, and full exciter voltage. On this basis a generator rheostat of usual design with cast-iron resistor grids will weigh approximately 25 lb per kilowatt of excitation.

For large generators, field rheostats usually are omitted and control is accomplished by a rheostat in the exciter field circuit.

29. Voltage Regulators. Generator voltage regulators are used to provide automatic control of the generator field excitation as required for voltage

control, frequency control, load control, power-factor control, and automatic synchronizing. For small generators, both a-c and d-c, the regulator operates on the generator field rheostat; but in large generators the field current is too large for economical regulator design and the regulator always operates on the exciter field rheostat.

Earlier types of regulators were of the vibrating or Tirrill type. A number of contacts on the regulator shunted sections of the exciter field rheostat so that when closed they raised the voltage to maximum and when open they lowered the voltage to minimum. The contacts were continuously vibrated in unison by a d-c magnet connected to the exciter in such a way that an increase in excitation voltage would open the contacts. Their movement was biased by an a-c magnet so connected to the generator that an increase in generator voltage would decrease the percentage of time that the contacts were closed and increase the time they were open, thus controlling the average excitation to meet the generator demands. The number of contacts vibrating in unison depended on the size of the exciter field. With this type of regulator a great deal depended on the performance of the contacts because, if they should fail to close, the voltage would drop to the minimum, and if they should fail to open it would rise to the maximum. Even with shunting condensers, the contacts required very careful supervision and frequent dressing.

The modern regulator, used almost exclusively in hydroelectric generation, is the nonvibrating, rheostatic or face-plate type. The rheostat arm is moved by a motor to the position demanded by the generator. In one type the arm is of sector shape with rolling contact and is operated directly by a small torque motor energized by the generator potential transformers. Its light construction and small movements result inherently in very quick response. Another type has a conventional rheostat arm operated by a reversing motor which is controlled by contacts opened and closed by a small torque motor energized by the generator potential transformers. Quick response is secured by two field-forcing contactors which operate on sudden demand for a large change in excitation, either to shunt the field rheostat or to open it until the slow-moving arm has time to arrive at the desired position.

Performance characteristics of a voltage regulator usually specified are sensitivity and speed of response. The sensitivity of a modern regulator is such that the normal contacts respond to a generator voltage change of 0.5% and the high-speed contacts can be adjusted to respond to a voltage change of 2.5 to 5%. The time of response of a modern regulator, measured from the time of a-c voltage change to the beginning of movement of the rheostat arm or to complete closure or opening of the field-forcing contactors, is not more than 5 cycles (on 60-cycle-per-second basis) with a change in voltage of not more than three times the high-speed sensitivity setting.

The a-c voltage which controls the voltage regulator must be representative of the a-c system being regulated. A single phase is not sufficient since with line-to-ground faults some of the phase voltages are reduced while others

may be increased. One type of regulator samples the 3-phase voltage through a network which delivers to the regulator a single-phase voltage representative of the average 3-phase conditions. Another type uses a 3-phase torque motor which feels the 3-phase voltages and delivers an average torque. In some regulators the a-c voltage, before going to the regulators, is rectified to direct current so that its influence on the regulator will not be affected by a change in frequency which sometimes results from system disturbances and especially from generator runaway speed.

Line-drop compensation sometimes is used to maintain a constant voltage not at the generator terminals but at a distribution center at the end of a transmission line. In this case the a-c voltage supply to the regulator is biased by the amount of current flowing so as to compensate for the voltage drop due to resistance and reactance of the line. Cross-current compensation usually is required when generators are operated in parallel so as to divide the kilowatts and kilovars between the various generators as desired. See Section 18.

The motor-operated rheostat, the contactors, and the field breaker and discharge resistors usually are mounted in the vicinity of the exciter so as to permit a short exciter field circuit. The voltage-sensitive element and relays usually are mounted on the main instrument board where they are under the supervision of the operator.

Other types of regulators are in limited use for special applications in hydro stations. A simple form of voltage regulator often is provided for the direct-connected pilot exciter to limit its voltage, in case of generator runaway, by inserting a block of resistance in the pilot exciter field circuit. The carbon-pile regulator utilizes the adjustable resistance obtainable from a series of carbon disks by varying the pressure between disks. The "Silverstat" regulator shunts small sections of the field rheostat by means of multiple silver contacts. Electronic exciters, rotating amplifiers, and static-type regulators are proving equally reliable, somewhat comparable in cost, and in some respects superior in performance to the more conventional exciters and regulators [12, 13, 14, 15].

30. Transformers. A transformer is defined as a device for transferring electrical energy from one a-c circuit to another a-c circuit, usually of a different voltage. The essential features are (1) a primary winding which receives energy from the supply circuit, (2) a secondary winding which delivers energy to the receiving circuit, and (3) an iron core common to both windings. The windings are so arranged on the core that alternating current in the primary from the supply circuit induces a magnetic flux in the core, which in turn induces an alternating current in the secondary.

The high-voltage winding is the one having the greater number of turns, and the low-voltage winding is the one having the lesser number of turns. The ratio, unless otherwise specified, is always understood to mean the turn ratio. If the transformer were ideal, that is, without losses or reactance, the voltage ratio would be equal to the turn ratio and the current ratio would

be the reciprocal of the turn ratio at all loads. Owing to the transformer losses and reactance, however, the primary voltage and current are higher than the values obtained by applying the turn ratio to the secondary voltage and current.

A constant-potential transformer, that is, one that at all loads has a constant voltage ratio, except as slightly modified by regulation, is the type that is commonly used for transmission and distribution of power. Sizes below 500-kva single-phase and 450-kva 3-phase are classed as distribution transformers, and those of larger size as power transformers.

A single-phase transformer, as distinguished from a polyphase transformer, has a single primary winding and a single secondary winding. It can be used by itself in a single-phase system or banked with similar transformers in a polyphase system. The 3-phase transformer is the only common polyphase type. It has three primary and three secondary windings arranged on a common core, and is used by itself in a 3-phase system. Advantages of the single-phase transformer are as follows: (1) banks of larger size can be shipped; (2) a defective transformer can be replaced more easily; (3) a single unit will suffice as a spare for several banks; and (4) by various groupings many different kinds of transformations may be obtained. Advantages of the 3-phase transformer are as follows: (1) the external connections are simpler; (2) less floor space is required; (3) the total weight is less; and (4) the first cost is less. A 3-phase transformer weighs from 5 to 15% less and occupies about 50% less floor space than a bank of three single-phase transformers having the same bank rating.

A 3-phase delta-connected shell-type transformer with one phase damaged may be operated in open delta at 58% capacity by short-circuiting both high-voltage and low-voltage windings of the damaged phase. If the transformer is of the core type, it cannot be operated in open delta unless the high- and low-voltage windings of the damaged phase can be open-circuited.

All transformers, with the exception of furnace transformers having bus-bar leads brought out through the cover and some other special transformers, are suitable for outdoor installation. For installation indoors where adequate ventilation can be provided, but where an oil-immersed transformer would constitute a fire hazard, three alternate types are now available for station service, namely: noninflammable-liquid-immersed transformers; dry-type, air-immersed transformers; and dry-type air-blast transformers.

31. Construction. Transformer cores are built up of thin sheets or laminations of special transformer steel having high magnetic permeability. To prevent eddy-current losses, the laminations are varnished before assembly, or otherwise insulated from each other and from the bolts which bind them together. The core is built up into one of two general types, the core type and the shell type. Each type has particular advantages in winding space, insulation, and bracing; the choice depends upon the capacity, voltage, and frequency.

The windings are of two general types: cylindrical windings, assembled concentrically; and discoidal windings, assembled interleaved. The choice depends upon the capacity, voltage, and frequency. Often the two types are combined in various ways. The relative grouping of the primary and secondary coils determines to a large extent the reactance of the transformer. It is essential that the windings be firmly braced to resist severe mechanical stresses without impeding the free circulation of oil about the conductors.

Insulation is one of the most important features of transformer construction. The insulation must withstand not only normal operating voltages, with an adequate margin of safety, but in addition the surge voltages to which it may be subjected [5 and 9]. The individual conductors of power and distribution transformers usually are wrapped with several layers of paper tape to provide insulation between turns. After winding, the coils are vacuum-dried and impregnated with insulating compound or varnish In assembly, the coils are insulated from each other by spacers and sheets of insulating material. They are insulated from the core and other windings by means of tubes, sheets, angles, channels, and collars of insulating material. Some manufacturers use a system of shielding the windings of power transformers to eliminate undesirable oscillations set up by surges. A static plate or shield usually is placed adjacent to the face of the line coil and connected to the line lead. This minimizes the turn-to-turn stresses by introducing the surge simultaneously to all turns in the line coil.

Bushings for bringing the leads out through the case are of three general types the solid type, the oil-filled type, and the condenser type. The solid type, used up to and including 34.5 kv, consists of a tube of porcelain, usually with petticoats on the part outside the transformer case. The oil-filled bushing, which is adaptable for voltages above 34.5 kv, consists of a tube made up of a number of concentric pieces of porcelain, filled with oil and containing a conducting rod or tube through its center. The condenser type, which is used for voltages of 13 8 kv and above, consists of a tube or round stud wrapped with treated paper with layers of metal foil at intervals. The successive layers, being larger in diameter and shorter in length toward the outside of the bushing, distribute the stress evenly over the entire radial thickness of the paper between the inner tube or stud and the outer ground flange of the bushing. At voltages of 66 kv and above care must be taken to prevent corona at the bushings both on the inside and outside of the case. All bushings should have a puncture voltage higher than the flashover and should have a flashover time lag exceeding that of the lightning arrester or protective gap used to protect the transformer. The flashover voltage at lightning frequency is practically the same either wet or dry and is about twice the flashover voltage at normal frequency.

The case or tank is generally built of boiler plate welded and tested to insure oiltightness. The tank usually has a top cover which is removed for lifting out the core and coils. A few transformers have been built with a joint at the bottom so that the tank can be lifted off, leaving the core and

coils standing on the bottom plate and base frame. For transformers too large for shipping clearances, the portion of the tank above the core and coils is removed and replaced by a flat cover for shipping. It is preferable to have the joint above the upper radiator connections. Tanks and radiators should be strong enough to withstand safely the oil pressure plus the inert gas pressure and to withstand vacuum while being filled with oil. Distribution transformer tanks are either smooth, fluted, or equipped with tubes to increase the radiating surface. Self-cooled power transformers have tubes or radiators welded or bolted to the tank to increase the radiating capacity of the tank. Water-cooled transformers have smooth tanks and internal cooling coils through which water is circulated. The cooling coils usually are arranged to drain by gravity when the water supply is shut off so that there will be no danger of freezing. The coils are made large enough so that the core and coils may be removed without disturbing them, or they may be attached to the cover so that they are removed with it. Power transformers should be provided with wheels for moving on rails or floor, jack lugs for lifting the complete unit, lifting eyes on core and coil assembly, oil drain valve, sampling valve, filtering connections, oil gage, oil thermometer, and hot-spot-indicating equipment when desired.

Conservation of the oil to exclude moisture and oxygen is of great importance. Moisture very greatly reduces the insulating properties of the oil, and the presence of oxygen, together with high operating temperatures, causes the oil to sludge. With changes of load, the resulting expansion and contraction cause breathing, and various methods have been developed for eliminating moisture and oxygen from the air breathed into the case. The chloride-of-lime breather has been used extensively. If recharged frequently, it is quite effective. The extensively used conservator or expansion-tank transformer is arranged to breathe through a conservator or expansion tank located above the main tank and connected to it by a small pipe. The main tank is completely filled with oil, the only oil surface exposed to air being that in the small tank. Circulation between the main tank and the small tank is so restricted that the oil in the small tank remains cool and therefore sludging is prevented. The inert gas transformer is filled above the oil with an inert gas such as nitrogen which is maintained at a slight pressure above atmospheric. With rise of pressure by temperature rise, a small amount of gas escapes from the transformer through a small breather, and with a fall of pressure below a predetermined pressure a small amount of inert gas is discharged into the transformer through a reducing valve from a storage cylinder. By this process not only is the atmosphere above the oil maintained inert but also any oxygen in solution in the oil gradually is replaced and sludging is eliminated.

Relief valves or diaphragms are usually provided to open or blow out in case of excessive internal pressure due to flashing or burning inside the case.

32. Rating. The rated output of a transformer is the product of the volts and amperes at its secondary terminals when delivering its maximum continuous load, and is expressed in kilovolt-amperes. The maximum continuous

output is limited by the heating. The Standards of the American Institute of Electrical Engineers specify the limiting temperatures for various classes of insulation and the methods of measuring the temperatures. For a 3-winding transformer, used for connecting three systems, the rating is stated specifically in terms of the input or output of each of the three windings.

The rated primary voltage, as defined in the Standards of the A.I.E.E., is the rated secondary voltage multiplied by the turn ratio. It follows that, when the transformer is carrying load at rated secondary voltage, the primary voltage is higher than rated by an amount equal to the regulation. The test voltage specified in the Standards of the A.I.E.E. is twice the normal voltage of the circuit to which the transformer is connected plus 1000 volts at not less than rated frequency and for a period of 60 sec. [5].

33. Taps and Internal Connections. Taps are usually provided to counteract line drop and transformer regulation. They are convenient for the operation of the system but from the construction standpoint are somewhat hazardous and expensive, especially in high-voltage transformers, and therefore should be specified only when actually required. Full-capacity taps are generally desirable, but their use requires that the windings have current capacity to supply rated kva at lowest tap voltage. Where reduced-capacity taps can be used, the cost of the transformer can be somewhat reduced. A tap-changing switch, whereby the taps can be selected, when the transformer is de-energized, by means of a handle outside the transformer case, is quite essential where system operation requires frequent changing of taps. Circuit-breaker equipment for changing taps under load, arranged for manual or automatic control, can be built into the transformer if required, but at considerably greater cost. Both primary and secondary windings are often arranged in groups which can be connected either in series or in parallel to give a selection of voltages. This feature is sometimes desirable for operating a system at half voltage initially, before the load is fully developed. The name plate should show clearly all taps and internal connections for which the transformer is designed.

34. Parallel Operation. Parallel operation of two or more transformers or banks of transformers, with distribution of the total load in proportion to their ratings, requires that they have (1) the same polarity and, if 3-phase, the same angular displacement, (2) equal turn ratios, (3) equal percentage impedances, and (4) equal ratios of reactance to resistance. Polarity and angular displacement are standardized [5]. If the ratios are not equal, they may be corrected by use of auto-transformers connected in series. If the impedances are not equal, they may be corrected by use of external impedances connected in series. Such devices, however, are expensive and are used only in emergencies. New transformers usually can be designed to operate in parallel with old ones if the necessary data on the existing units are available to the manufacturer.

35. Resistance, Reactance, and Impedance. Resistance, reactance, and impedance of a transformer may be expressed in ohms, or as internal

voltage drops, in percentages of rated secondary voltage, caused by rated current at unity power factor. The relation between the percentage drop and the ohmic drop is:

$$\frac{\text{Per cent drop}}{100 \times \text{Ohms}} = \frac{\text{Rated current}}{\text{Rated voltage}} = \frac{\text{Rated volt-amperes}}{(\text{Rated voltage})^2} \qquad [4]$$

If the resistances of the primary and secondary windings are given separately, the primary resistance may be converted to equivalent secondary resistance by multiplying it by the square of the reciprocal of the turn ratio. The equivalent resistance of the primary can then be added to the secondary resistance to find the total transformer resistance in terms of secondary ohms. By a similar process the secondary resistance can be converted to equivalent primary resistance and added to the primary resistance to give the total transformer resistance in terms of primary ohms. For example, a transformer having a turn ratio of $\frac{1}{6}$, a primary resistance of 2 ohms, and a secondary resistance of 70 ohms, has a total resistance expressed in terms of secondary ohms of:

$$70 + 2 \times \frac{36}{1} = 142 \text{ ohms}$$

and expressed in terms of primary ohms of:

$$2 + 70 \times \frac{1}{36} = 3.94 \text{ ohms}$$

Reactance and impedance may be converted similarly, but it is seldom necessary to consider the reactance or impedance of either winding separately.

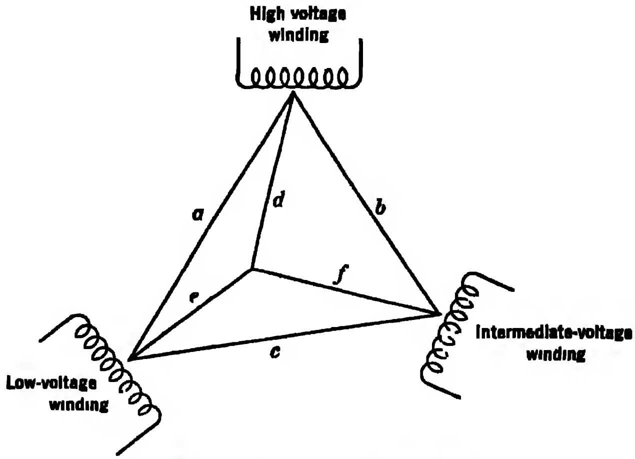
Low resistance is desirable in order to reduce losses, and low reactance is desirable in order to give good regulation. In connection with large power systems, however, it is sometimes desirable to build transformers with comparatively high reactance in order to reduce short-circuit currents.

36. Regulation. The regulation of a transformer is the difference between the no-load secondary voltage and the rated-load secondary voltage expressed in percentage of rated secondary voltage, the primary voltage being constant at such value as to give rated secondary voltage at rated load. The regulation varies with the power factor of the load, and therefore the power factor at which the regulation is given must always be stated. The regulation may be obtained by measuring the change in voltage with change in load, or by measuring the resistance and reactance and computing the regulation from the equation:

$$\text{Per cent regulation} = q_r \cos\theta + q_x \sin\theta + \frac{(q_x \cos\theta - q_r \sin\theta)^2}{200} \qquad [5]$$

where q_r = per cent resistance drop;
q_x = per cent reactance drop;
$\cos\theta$ = power factor;
$\sin\theta$ = reactive factor.

If a transformer has more than two windings, the determination of its regulation, when loads are taken from two or more windings simultaneously, is more complicated. The impedances between windings must be separated into the parts belonging to each winding. Equations for separating the reactances for a 3-winding transformer when the reactances between the three pairs of windings are known are given in Fig. 7. The reactances be-



a = reactance from HV to LV in percentage
b = " " HV to IV " "
c = " " LV to IV " "

$$d = \frac{a+b-c}{2}$$

$$e = \frac{a+c-b}{2}$$

$$f = \frac{c+b-a}{2}$$

FIG. 7 Method of separating reactances of a three-winding transformer.

tween pairs of windings must be on a common kva basis. The per cent effective resistance drops may be separated in the same way. The regulation for any combination of loading may be calculated by applying Eq. 5 to each branch, *d*, *e*, and *f*, at the kva in that winding and then adding. This method of obtaining regulation is approximate, but the error is small.

37. Efficiency. The efficiency of a transformer is the ratio of the kilowatt output to the kilowatt input. The losses are classified as no-load losses, or those that continue as long as the transformer is energized regardless of whether it is delivering any load, and load losses, which are dependent on the load delivered. The Standards of the American Institute of Electrical Engineers specify that the conventional efficiency of a transformer shall be determined as follows:

No-load losses shall include the core loss, the I^2R loss due to the exciting current and the dielectric loss in the insulation at 75° C. They shall be measured with open secondary circuit at rated frequency, and with an applied primary voltage, giving the rated secondary voltage plus the IR drop which occurs in the secondary under rated load.

Load losses shall include I^2R losses, and stray-load losses due to eddy currents caused by fluxes varying with load at 75° C. They shall be measured by wattmeter by applying a primary voltage, at rated frequency, sufficient to produce rated load current in the windings, with the secondary windings short circuited.

With the losses given, the efficiency may be computed from the equation:

$$E = \frac{100P}{P+L} = 100 - \frac{100L}{P+L} = 100 - \frac{100(I+C)}{P+(I+C)} \qquad [6]$$

where E = efficiency in per cent;
P = delivered power in kilowatts;
L = losses in kilowatts;
I = no-load losses (commonly called iron losses) in kilowatts;
C = load losses (commonly called copper losses) in kilowatts.

It should be noted that, with a fixed value of kva, P is directly proportional to the power factor, I is independent of load or power factor, and C is proportional to the load but independent of the power factor. It is necessary, therefore, when giving the efficiency of a transformer, to state the load and power factor at which it applies. It is usual for the manufacturer to submit with his quotation the efficiencies at several loads and at unity power factor. The full-load, unity-power-factor efficiency of a modern 60-cycle power transformer varies from 96% to 99.5% depending on the rating. A 25-cycle transformer is slightly less efficient.

By properly proportioning the iron and copper losses, the manufacturer can usually obtain the highest efficiency at 50, 75, or 100% load, as desired, to give the best all-day efficiency. For instance, a transformer which is to carry full load all the time should have its best efficiency at 100% load, which means that the iron losses and the copper losses should be about equal at full load, while a transformer which is to carry only partial load many hours of the day should have its best efficiency at less than 100% load, which means that the iron losses and the copper losses should be about equal at less than full load.

38. Cooling. The losses of the transformer appear as heat, which must be carried away as fast as generated so as to prevent excessive temperature rise. The method of cooling classifies the transformer in one of four general types which are, in order of decreasing importance: self-cooled, water-cooled, forced oil-cooled, and air-blast.

The self-cooled transformer has the core and windings immersed in oil, and the case is either fluted or provided with radiators so that the oil is cooled by natural radiation. The oil circulates by convection upward through the hot windings and downward through the flutes or radiators. This type can be built in practically any commercial capacity and voltage.

Fans or blowers can be applied to the radiators for more effective cooling, but it should be remembered that such forcing generally increases the gradient between the hottest-spot temperature and the observable temperature. Consequently, forced cooling should be used with caution if the life of the insulation is not to be endangered. Ordinarily the continuous capacity of a transformer can be increased 25% and the short-time peaking capacity 33⅓% by the addition of blowers, without undue risk of shortening the life of the insulation. Blowers often are applied to transformers used for stepping up the output of hydro generators. The transformers are then so rated that the self-cooled capacity is sufficient for the generator normal output and the air-blast capacity is used only part time during high-head conditions (see Section 4).

The water-cooled transformer is also oil-insulated but the tank is plain. Around the inside of the tank at the top, where the oil is hottest, a cooling coil is placed, through which cold water is forced. Cooling water at a hydroelectric generating station is usually available in such quantities that the discharged water can be rejected economically. At the substation, however, water is usually too expensive to waste and must be cooled in a cooling tower or spray pond and recirculated. It is desirable to use a low water pressure to prevent the possibility of water leaking into the oil, and for this reason it is customary to discharge the water from the transformer at atmospheric pressure.

The forced oil-cooled transformer is similar to the water-cooled, except that it has no water coils. The oil is circulated by pumps through external radiators which are cooled by air, or through surface coolers which are cooled by water. Oil circulation should be upward through the transformer and downward through the cooler. Water circulation should be upward through the cooler. It is important that the oil pressure be maintained higher than the water pressure to prevent water leaking into the oil. Purity of the water is not as important as with the water-cooled transformer, for the cooler can be made in sections and part at a time can be cleaned and inspected. At the generating station it is sometimes possible to place the oil-cooling coils or radiators in the tailrace.

Individual cooling units have been developed for mounting on transformers in place of or in addition to the usual radiators. Each unit consists of a small oil pump, a radiator with blower, or a heat exchanger with water connections, and elbows for attaching to the transformer tank. The pump and motor are both entirely enclosed in a casing which forms a part of the oil piping, and there are no glands that might leak oil. These units are sometimes used to increase the capacity of an old transformer; the capacity may be increased up to 50%. The gradient between the hottest copper temperature and the observable oil temperature, however, is increased, and the transformer cannot be forced for more than a small percentage of the time without appreciably shortening the life of the insulation.

The amount of cooling water required for a water-cooled transformer or for a forced oil-cooled transformer, and the amount of oil circulation required for a forced oil-cooled transformer are given by the following equations, which do not take into account any radiation. For forced oil-cooled transformers the actual amount of water required may be found to be 10 or 20% less, owing to radiation from the oil piping and coolers.

$$Q_w = \frac{3.8P}{T_w} \tag{7}$$

$$Q_0 = \frac{4.5P}{T_0 S_0} \tag{8}$$

where Q_w = gallons per minute of cooling water required for water-cooled or forced oil-cooled transformers;
Q_0 = gallons per minute of oil circulated between transformer and cooler for forced oil-cooled transformers;
P = transformer loss in kilowatts;
T_w = temperature rise in degrees Centigrade between ingoing and outgoing water;
T_0 = temperature rise in degrees Centigrade between cool oil and hot oil, usually equal to 10 to 15;
S_0 = specific heat of oil, equal to 0.40 to 0.50.

39. Oil. Oil for transformers is obtained from crude petroleum by fractional distillation. Desirable qualities are (1) high resistivity and dielectric strength, (2) low viscosity, (3) high flash and burning points, (4) high thermal conductivity and specific heat, (5) chemical neutrality toward metals and insulating materials, and (6) chemical stability at high temperatures. Physical and chemical properties of desirable transformer oil are as tabulated, when tested in accordance with American Society for Testing Materials specification D-117.

Specific gravity	0.85 to 0.90
Viscosity (Saybolt Universal at 100° F)	55 to 64 sec
Flash point	130° C
Fire point	150° C
Pour point	−40° C
Neutralization number	0.05 mg
Dielectric strength	22 kv *
Mineral acids	None
Free and corrosive sulphur	None
Steam emulsion	30 sec

* Minimum after dehydration at destination shall be 28 kv.

Moisture, even in very minute quantities, greatly reduces the dielectric strength of oil. Before being put into service, the oil should be tested and should not break down at less than 22,000 volts. This requires that it con-

tain less than about 0.001% moisture by weight. It should be sampled about once a month while in service, and when the breakdown voltage falls to 17,000 volts it should be dried again.

The two common methods of drying and purifying oil are: (1) by passing it through a centrifuge, and (2) by forcing it through dry blotting paper. The former method effectively removes the heavy foreign matter and some of the water. In order to use the centrifuge to best advantage, the oil must be warmed, and, unfortunately, its tendency to hold moisture increases enormously with a moderate rise in temperature. The filter-press method removes all suspended matter, and, if proper attention is given to drying the blotting papers, thoroughly and frequently, in an electric oven, this method removes practically all traces of moisture. A combination of the two methods is very effective.

40. Transformer Specification. The following brief outline is intended as a guide to the principal items to be included in a specification for an important power transformer:

General. Number wanted; scope of work by purchaser and by contractor; transporting, handling, and erecting facilities; drawings required; standards of workmanship, test and performance, such as A.I.E.E., A.S.A., N.E.M.A., A.S.T.M., N.E.S.C.; shop inspection, assembly, testing, painting, packing [5].

Type and Rating. Step-up or step-down power type; outdoor; oil-immersed, noninflammable-liquid-immersed, dry type; number of windings; continuous kva output at 55° C temperature rise; phases; cycles; voltages; connections; voltage taps; bushing ratings; impedances; neutral voltage; neutral grounding reactor rating; transformer polarity if single-phase, phase-angle displacement if 3-phase.

Construction. Core material, treatment, assembly, grounding, rigid supports in tank, lifting lugs; winding materials, assembly, drying and impregnating, leads to terminal board, rigid supports; neutral to be brought out; filling with oil under vacuum; tank material, welding, double-welded seams where practicable, split for shipping, must withstand pressure and vacuum, manhole for entrance, guides for lowering core and coils, lifting and jacking lugs, base, wheels; oil drain and sampling valves, filter valves, pressure relief valve; tap-changing facilities; bushing and terminal types and details; desired bushing locations; low center of gravity.

Cooling Facilities. Self-cooled with or without fans, water-cooled, forced oil-cooled with blowers or heat exchangers, provision for fans later, air blast; radiators accessible for cleaning and painting, fixed or removable, shut-off valves, drains, vents; fan controls and thermostats, fan motor and control voltages; water-cooling coils to drain, facilities for cleaning, temperature of water available, water-flow gage, alarms; forced-oil pumps, motors, controls, voltages; air-blast arrangement.

Oil and Oil-conserving Facilities. Oil type and minimum requirements; conservator, inert gas or other equipment; automatic operation; alarms; shipment of oil in transformer or separately.

Accessories. Oil-level gage; low-oil-level alarm; oil temperature indicator; hot-spot temperature indicator; hot-spot temperature detector for remote recorder; bushing current transformers and short-circuiting terminal blocks; rating and connection plate; desired locations of accessories; voltage of alarm and auxiliary circuits; auxiliary wiring and terminals; tools; lifting beam; spare parts.

TABLE 6

STANDARD TRANSFORMER CONNECTIONS

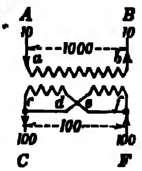
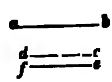
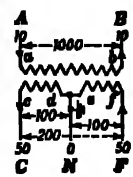
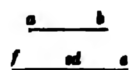
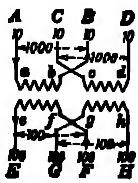
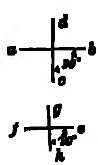
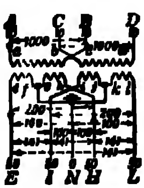
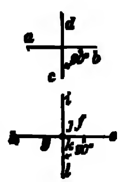
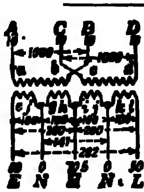
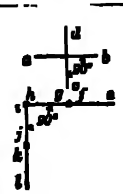
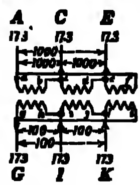
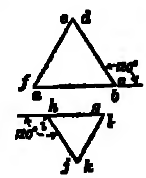
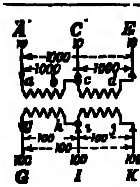
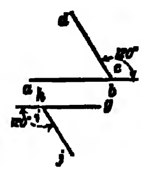
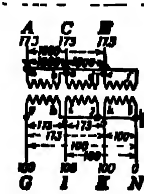
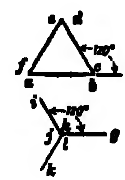
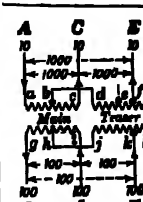
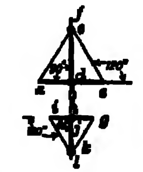
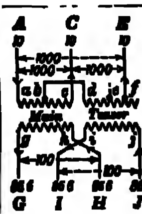
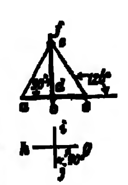
Description L = capacity of bank. T = capacity of each transformer.	*Diagram* Arrows indicate instantaneous currents. Figures indicate rins, volts and amperes.	*Vectors* Letters correspond to those in the diagrams.
Connection 1. Primary. Single-phase, 2-wire. Secondary. Single-phase, 2-wire, double winding connected in multiple. $L = T = 10$ kva.		
Connection 2. Primary. Single-phase, 2-wire. Secondary. Single-phase, 3-wire, neutral grounded, double winding in series. Known as Edison 3-wire system. No current in neutral if load is balanced. Neutral may be omitted for 2-wire circuit. $L = T = 10$ kva.		
Connection 3. Primary. Two-phase, 4-wire, non-interconnected. Secondary. Two-phase, 4-wire, non-interconnected $L = 2T = 20$ kva.		
Connection 4. Primary. Two-phase, 4-wire, non-interconnected. Secondary. Two-phase, 5-wire, interconnected, neutral grounded. No current in neutral if load is balanced. Neutral may be omitted for 4-wire interconnected circuit. $L = 2T = 20$ kva.		
Connection 5. Primary. Two-phase, 4-wire, non-interconnected. Secondary. Two-phase, 5-wire. No current in neutrals if load is balanced. Neutral wires may be omitted for 3-wire circuit. $L = 2T = 20$ kva.		

TABLE 6—*Continued*

STANDARD TRANSFORMER CONNECTIONS

Description L = capacity of bank. T = capacity of each transformer.	*Diagram* Arrows indicate instantaneous currents. Figures indicate rms. volts and amperes.	*Vectors* Letters correspond to those in the diagrams.
Connection 6. Primary. Three-phase, 3-wire, delta. Secondary. Three-phase, 3-wire, delta. Line current = transformer current × $\sqrt{3}$. $L = 3T = 30$ kva.		
Connection 7. Primary. Three-phase, 3-wire, open delta or V. Secondary. Three-phase, 3-wire, open delta or V. Current in each transformer 30° out of phase with transformer voltage, resulting in reduced capacity. $L = 2T \times 0.866 = 17.3$ kva.		
Connection 8. Primary. Three-phase, 3-wire, delta. Secondary. Three-phase, 4-wire, star or Y, neutral grounded. Neutral may be omitted for 3-wire circuit. May be operated with primary star and secondary delta. Star connection for both primary and secondary not recommended unless neutral is grounded. $L = 3T = 30$ kva.		
Connection 9. Primary. Three-phase, 3-wire, Scott or T. Secondary. Three-phase, 3-wire, Scott or T. Taps b and h = 50%. Taps e and k = 86.6%. Current in main transformer 30° out of phase with transformer voltage, and voltage in teaser transformer only 86.6% utilized, resulting in reduced capacity. $L = 2T \times 0.866 = 17.3$ kva.		
Connection 10. Primary. Three-phase, 3-wire, Scott or T. Secondary. Two-phase, 4-wire, non-interconnected. May be operated with primary 2-phase and secondary 3-phase. Tap b = 50%. Tap e = 86.6%. Three-phase windings operate at reduced capacity as with connection 9 and 2-phase windings designed for 13.4% less capacity to correspond. $L = 2T \times 0.933 = 17.3$ kva.		

Neutral Grounding Reactor. Rated voltage, reactance, current, temperature rise and time; resistance less than 10% of reactance, self-cooled, air-core type, oil-immersed in hermetically sealed tank; magnetic shield between winding and tank; shunt arrester.

Factory Tests for Each Transformer. Pressure test of double-welded seams with nitrogen, entire tank and radiators with oil and air; cold resistance of each winding; turn ratio; polarity; angular displacement; exciting current at rated frequency and at 100 and 110% rated voltage; excitation losses at rated frequency and at 90, 100, and 110% rated voltage; impedance between each pair of windings; zero and positive phase sequence if 3-phase; regulation at rated load and 1.0, 0.9, and 0.8 lagging power factor; load losses at rated frequency; dielectric tests.

Factory Test on One Transformer of a Lot. Temperature test at an equivalent to rated load; impulse tests when specifically required.

Guaranteed Performance Data by Bidder. Continuous kva output at 50° C rise by resistance; regulation at 1.0, 0.9, and 0.8 power factor; no-load losses at 90, 100, and 110% rated voltage; total losses at rated load and voltage; exciting current at 100 and 110% rated voltage; impedance; polarity if 1-phase, angular displacement if 3-phase.

Design Data by Bidder. Type designation; exceptions to specification; testing facilities available; outline, dimensions; untanking height and weight; bushing make, type, rating; bushing withstand voltage, wet, dry, impulse; construction of transformer, tap changer, neutral reactor; weights of core and coils, tank and fittings, oil, total; gallons of oil.

41. Connections. Of the large number of possible connections and groupings of transformers, the most commonly used are shown in Table 6

42. Installation. Installing transformers outdoors saves building space, obtains good ventilation, and reduces fire hazard. When installed indoors, transformers should be surrounded by substantial fire-resisting walls and, unless of the water-cooled type, should be provided with adequate ventilation to carry off the losses. It is desirable to leave clear space all around each transformer for inspection, reading of gages, and operation of valves. It has been usual practice to provide transformers for 115-kv voltage or 15-ton weight, or greater, with wheels and to mount them on short rails so that they can be moved onto a transfer car and under the station crane or a special hoist for untanking. The present trend, with more dependable transformers, is to make less provision for untanking, and, with very large transformers, to install them on skids instead of wheels. Piping from a central oil-filtering room or portable oil-filtering equipment usually is provided to maintain the oil, but periodic filtering seldom is necessary with inert-gas-filled transformers. Large transformers often are shipped in dry gas to save weight. At destination the gas is displaced by dry oil without exposing the windings to air or moisture. It sometimes is necessary, before voltage is applied, to warm the windings or even to rock the transformer to be sure that no gas pockets or bubbles remain in the windings.

43. Bibliography. (See also Chapter 40, Section 17 for general bibliography.)

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CHAPTER 42

SWITCHING, WIRING, AUXILIARY POWER, AND LIGHTING

By Raymond A. Hopkins *

1. Main Electrical Connections. One of the most important considerations in the electrical design of a hydroelectric generating station is its connection to the transmission system. Unlike most steam plants located close to the load, the hydro plant usually is distant from large load centers, often far away in the mountains. It follows that seldom is much of the output distributed at generator voltage. In fact, a hydro station usually is associated with long lines and high transmission voltages.

The scheme of bussing and switching on both the low-voltage side and the high-voltage side of the transformers depends on the number of outgoing lines, the number of transformers and generators, and the operating dependability and flexibility desired. The scheme should provide for: (1) operation of generating units at individual loadings which result in reasonably high efficiencies at all values of station loading; (2) retiring of generating, transforming, and switching equipment without rejecting load; (3) synchronizing of each generating unit individually; (4) isolation of faults within the station or on the transmission system; (5) minimizing of fault currents; (6) promotion of power system stability; (7) splitting of the power system under certain conditions of operation; and (8) expansion of the facilities as may be required during the life of the plant to accommodate additional generation, lines, and voltages, or greater operating dependability and flexibility. Obviously, a simple, straightforward, systematic arrangement is conducive to most of the above-named objectives.

A typical main electrical connection diagram of an important hydro station, Fig. 1, illustrates the features listed in the previous paragraph. The bussing on the high-voltage side of the transformers permits the operation of as many or as few of the generating units as may be desired to load them close to their best efficiencies. A generator-transformer circuit, up to and including the two high-voltage breakers, may be retired for inspection or maintenance without disconnecting the load. Either of the two high-voltage breakers of a circuit may be retired without rejecting load or generation. A line breaker may be retired by connecting the line through its motor-operated

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disconnect to the transfer bus and feeding it through one of the generator breakers connected to that bus; when this is done, the line relays are transferred from the current transformers on the line breaker to those on the generator breaker. Each generating unit can be synchronized by means of one of its high-voltage breakers. Separate relaying can be provided for relatively small zones such as each generator, each transformer, each bus section, each line, each station-service transformer, or each section of the auxiliary-power bus. Heavy concentration of fault current on the generator-voltage circuits and on the auxiliary-power bus is prevented by avoiding any bussing at generator voltage and by interlocking the three main breakers on the auxiliary-power bus. Power system stability is aided by bussing the

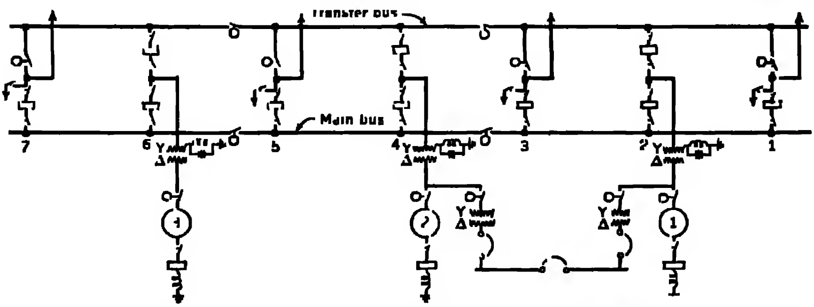


FIG. 1. Typical main electrical connection diagram.

lines. The power system can be split if required for certain conditions of operation by opening a bus disconnect; and during this operation the generating capacity can be divided as desired between the two parts of the system. The arrangement can be made flexible and expandable by judicious arrangement of the switchyard and design of the switching structures (see Section 15).

Many other bussing and switching schemes are in use, designed to meet predominant requirements of specific power systems. The foregoing discussion illustrates the type of problems involved and a satisfactory solution.

2. Switching Equipment. Switching equipment consists of switches, breakers, and other devices used by the station operators for opening or closing electrical circuits and connecting or disconnecting generators, transformers, and other equipment. In most cases switches are fused and circuit-breakers are equipped with trip coils or relays so that they will automatically open their circuits when disturbances occur on the system, and can thus be used both as switching equipment and as protective equipment.

Low-voltage (0- to 600-volt) switching devices used for controlling and protecting lighting, auxiliary-power, and signal circuits throughout the station are usually in the form of knife switches, snap switches, fuses, and small circuit-breakers. They are mounted on panels or in cabinets and operated manually, mechanically, or electrically.

Generator-voltage (2400- to 25,000-volt) switchgear, used for controlling and protecting the main low-voltage generator and transformer circuits, includes manually and motor-operated disconnecting switches, fuses, and electrically or pneumatically operated oil and air circuit-breakers. If used indoors it is inclosed in steel or masonry compartments to isolate it from other equipment and from the station personnel. For outdoor use it is made weatherproof and mounted on concrete bases or steel structures.

Transmission-voltage switchgear, used for transformer and transmission line circuits, consists of manually or electrically operated disconnecting switches, fuses, and electrically or pneumatically operated breakers almost universally mounted outdoors in the switchyard.

3. Knife Switches. Knife switches are available in ratings of 0–600 volts and 0–3000 amperes, and in a variety of forms such as single- and multiple-pole; single- and double-throw; front- and back-connected; top- and bottom-fused and unfused; and punched- and milled-clip. They generally are mounted on switchboards or panelboards. They always should be mounted vertically with hinges at the bottom, except that double-throw switches should be mounted horizontally. The blades should be "dead" when open. Switches are used very extensively in the generating station and the substation for controlling excitation, auxiliary-power, lighting, and signal circuits. Plain knife switches are not intended to be opened under load but, if equipped with quick-break and arc-reducing devices, may be used to open rated current. Knife switches are rated on the basis of the maximum current they can carry continuously with a temperature rise of not more than 30 degrees Centigrade.

Silver plating to a thickness of about 3 mils is applied by some manufacturers to moving electrical contacts, whether operated in air or under oil. The greater conductivity of silver than of copper, and particularly the high conductivity of silver oxide, assures permanently low resistance of the contact with much less area and pressure than is required for equal performance with copper-to-copper contacts.

4. Fuses. A fuse is the simplest device for automatically opening a circuit on overcurrent. In its simplest form, it consists of a metal wire or strip of such composition and cross-section that it melts on excess current.

Low-voltage fuses are of plug type rated 0–30 amperes, 125 volts, and cartridge type rated 0–600 amperes, 250 and 600 volts. The cartridge fuse is packed with a heat-resisting powder to cool the arc. Link fuses occasionally are used in ratings 600–1500 amperes, 125 and 250 volts, in locations not accessible to the public. All these low-voltage fuses are designed to carry their rated current indefinitely and to blow in from 1 to 5 minutes on currents in excess of 115% of rating. They are used to some extent on lighting and auxiliary-power circuits, particularly for such locations as cranes where small breakers might be adversely affected by vibration. A definite application is for short-circuit protection of auxiliary-power motors, since they open on short circuit considerably faster than breakers or contactors. Various types of renewable fuses and time-lag fuses are available for applications where

fuses are used for running protection as well as short-circuit protection, but the one-time fuse usually is more economical when used only for short-circuit protection. Low-voltage fuses have interrupting ratings up to 10,000 amperes.

High-voltage fuses are available for voltages up to 138,000 volts and interrupting capacities up to 1,500,000 kva. They are of various types, such as link, powder-filled cartridge, oil-filled, arc-quenching, liquid-filled with retracting spring, boric acid expulsion, and others. All have inverse-time characteristics usually based on an ambient temperature of 20 or 25° C. They are used for protecting potential transformers, small power transformers, and distribution branch circuits. Mountings are available which will automatically replace blown fuses with new ones two or three times before requiring attention.

5. Inclosed Switches. Knife switches inclosed in steel boxes arranged to be operated from the outside, commonly known as "safety switches," are extensively used throughout the station at motors, on cranes, and at other places apart from the switchboards. They are fused or unfused as desired. The box is generally constructed so that the switch must be opened before the fuses can be reached and so that it can be locked in the open position. Both these features greatly safeguard the maintenance men.

6. Field Switches. The field switch is designed with a discharge clip which closes while the switch is being opened, so as to discharge the inductive energy from the field through a field-discharge resistance. It is sometimes provided with arcing tips and barriers. The field-transfer switch is a double-throw switch with extra long clips arranged so that the field can be thrown from one bus to another without being opened.

7. Disconnecting Switches. Disconnecting switches are principally used for isolating oil circuit-breakers, generators, transformers, arresters, and other equipment; as selector switches in conjunction with double buses and single circuit-breakers; and as tie switches between circuits. They are not designed to be opened under load. The simplest disconnecting switch consists of a single-pole knife switch arranged with an eye to receive a switch hook by means of which it is opened and closed. It is mounted on porcelain insulators and a suitable metal base. A latch to prevent the switch from flying open under short-circuit current is usually necessary except on very small power systems.

Gang-operated disconnecting switches have the following advantages: (1) when using them it is impossible to operate inadvertently two phases of one circuit and one phase of an adjacent circuit; (2) they can be operated conveniently without a switch hook; and (3) they can be interlocked with breakers or other disconnects. Gang-operated switches often are motor-operated from the control switchboard.

For outdoor use, disconnecting switches are sometimes provided with horns to direct the arc upward and are then capable of breaking the charging current on short lines or small transformer banks. At 60,000 volts this has

been done successfully on lines up to 60 miles long and on transformers up to 7500 kva.

8. Magnetic Switches. Magnetic switches or contactors are used extensively for starting and stopping motors and for controlling circuits from a distance. They are held closed by the closing solenoid, and they fall open by spring action and gravity when the solenoid is de-energized. They usually are provided with overcurrent protection of the thermal or the magnetic type or in some instances a combination of both. Contactors are not intended for high interrupting duty. For power-station application they should have fuses or breakers ahead of them to trip on heavy short circuits, unless the contactors are far enough from the power source so that the intervening circuit can be depended upon to reduce the short-circuit current to within the contactor, interrupting, and short-time thermal ratings.

9. Air Circuit-breakers. Air circuit-breakers are used on excitation, battery, motor-generator, and auxiliary-power and lighting circuits. They are available in voltages up to 750 and in ampere capacities up to 10,000.

The main contact usually consists of a laminated copper brush bearing against cast-copper blocks or of a silver-plated copper moving element bearing against silver-plated copper blocks. The secondary and arcing contacts consist of replaceable carbon, graphite, or metal blocks. These secondary and arcing contacts close before and open after the main contacts, thus protecting the main contacts from the arc. Breakers of this type are rated on the basis of the maximum current they will carry continuously with a temperature rise on the contacts and terminals, by thermometer, of not more than 30° C.

Air breakers can be arranged to operate either manually or electrically. Operating coils can be wound for 125 or 250 volts and should be designed for satisfactory operation at any voltage between 50% and 120% of the normal operating voltage, to take care of variations in the control-battery voltage. The breaker is held closed by a latch which can be tripped either manually or electrically.

Automatic features include overload, underload, reverse-current, and undervoltage trips. Auxiliary switches are usually provided for operating alarm bells, indicating lamps, and interlocks.

Some breakers are arranged with a separate operating handle on each pole, and some are arranged to trip free from the operating handle. One or the other of these devices or a series knife switch should always be used so as to prevent the possibility of closing the breaker forcibly against a short circuit.

Air-blast breakers are breakers in which the contacts are parted by action of compressed air against pistons and a blast of compressed air also is directed across the arc path to extinguish the arc. They are available in capacities sufficient for the generator-voltage switching of most modern hydro stations; a few are in use. They are being developed for heavy-duty transmission-voltage switching with a view particularly to reducing speed of opening and avoiding the fire hazard of oil. A few are in operation at these voltages.

10. Oil Circuit-breakers. Oil circuit-breakers, which open their contacts under oil, are used principally for controlling alternating-current circuits. They may be obtained in ampere capacities from 50 to 4000; for voltages from 440 upward; with 1, 2, 3, or 4 poles; manually, electrically, or pneumatically operated; and for indoor or outdoor service. The selection of the proper breaker for each circuit in a power system involves consideration of several very important factors, such as normal and abnormal voltage, normal and abnormal current, interrupting duty, method of operation, arrangement of terminals, space available, accessibility for repairs, altitude above sea level, and temperature.

A breaker is rated in accordance with the Standards of the American Institute of Electrical Engineers on the basis of (1) the normal rms voltage of the circuit on which it is to operate, (2) the normal frequency of the current, (3) the normal rms current it is to carry continuously with a temperature rise of the oil and contacts by thermometer of not over 30° C above an ambient temperature of 40° C, (4) the short-time rating, based on the maximum rms current the breaker will carry safely for a short time, as 1, 2, 3, or 5 sec, without excessive heating or mechanical injury, (5) the maximum rms current against which the breaker can be closed momentarily without damage, and (6) the maximum rms current at normal voltage which it can interrupt under prescribed conditions at stated intervals a specified number of times.

The value of the short-circuit current which a breaker will be called upon to interrupt in a particular location on a system depends upon the connected synchronous capacity of the system, the characteristics of the machines, transformers, circuits, and other equipment, and the time allowed for the breaker to open after the short circuit occurs. (See Chapter 40, Section 13.)

In the smaller breakers the three poles are arranged in a single oil tank; in medium and large breakers each pole has a separate tank. Auxiliary contacts are provided to open after and close before the main contacts so as to relieve the main contacts of burning action of the arc.

Two principal methods at present are in use to extinguish the arc, both depending upon the fact that when an arc is formed under oil the heat generates a bubble of gas which violently expands. The deion method provides multiple small slots into which the oil is forced, thus allowing the gas to deionize and prevent re-establishment of the arc after a current zero. The oil-blast method provides a chamber around the contacts with outlets so arranged that the pressure generated from the gas bubble forces a blast of cold oil between the contacts which prevents re-establishment of the arc after a current zero. High-interrupting-capacity breakers generally use both of these methods.

Reclosing breakers are designed to reclose automatically from one to three times after being tripped by a fault on the circuit. Since some faults disappear after one or two openings and reclosures, such a breaker can be used to restore service after only a few cycles of outage. The breakers, however,

are derated in interrupting capacity after the first interruption and must be selected on this basis.

Accessories for an oil circuit-breaker include hand-closing lever; tank-lifting and tank-removing device; control relay for reducing the duty on the control switch; bushing current transformers and, for higher voltages, bushing potential devices for relaying, metering, and synchronizing; auxiliary switches for controlling pilot lamps, alarms, and interlocks; oil gages; oil-drain and sampling valves; and filter-press connections. Oil is similar to that used in transformers (see Chapter 41, Section 39).

Breakers may be mounted on switchboard panels, but this practice is not recommended for voltages higher than 2500 or for ampere ratings higher than 800. Larger breakers are mounted away from the switchboard on pipe or steel framework or in fireproof housings. The last method is used for the main circuits of all but the smallest of power stations and substations. For voltages above 25,000 and occasionally for lower voltages, breakers are located outdoors where space is available for greater clearances.

11. Oil Circuit-breaker Specification. The following brief outline is intended as a guide to the principal items to be included for a specification for an important high-voltage oil circuit-breaker:

General. Number wanted; drawings required; standards of workmanship, test, and performance, such as those of the A.I.E.E., A.S.A., N.E.M.A., A.S.T.M., and N.E.S.C.; shop inspection, assembly, testing, painting, packing. (See A.S.A Standard C37.4 to .9.)

Type and Rating. Outdoor or indoor; one tank per pole; electrically or pneumatically operated; floor- or frame-mounted; rated voltage, continuous current, interrupting capacity, opening time, reclosing time.

Tanks and Fittings. Low-carbon steel, suitable for welding; oiltight single- and double-welded joints; lifting jacks; provision for preventing gases from passing from tank to tank or to mechanism housing; access manhole for floor-mounted tanks; emergency gas vent; drain and filter connections and valves; oil gages; floor clamps; cleats in bottom of tank and on top to safeguard workmen from slipping.

Contacts. Adjustable to allow for wear; replaceable; minimum of moving parts; easily accessible for inspection and replacement.

Bushings. Wet-process porcelain; homogeneous, glazed; puncture strength greater than dry flashover; taps for potential networks if 110 kv or over; liquid-level indicators, drains and vents if liquid-filled; oil in bushing same as in tank; type of terminals

Operating Mechanism. To close and open breaker by remote electric control; all three poles to be operated by one mechanism; operating mechanism to be inclosed in weatherproof steel housing with access doors on three sides and with thermostatically controlled electric heaters; local close and open control switch within housing; trip lever outside housing, guarded against accidental tripping; manual closing device trip-free if it is possible to close breaker by one sweep of handle; provision for automatic reclosing; fast, positive; no objectionable rebound or critical adjustment, contacts of three poles adjustable to open and close simultaneously; control circuits required to carry not more than 5 amperes to close or 200 amperes to open; 10-stage auxiliary switch, each stage adjustable to open or close and to operate at any point in the breaker operation; provision for attaching operation analyzer; switches and fuses for control circuits and

heaters; terminal blocks for all control and power-supply wiring; short-circuiting blocks for current transformers; all wiring stranded; removable plate in housing for drilling for conduits; operation counter visible with housing doors closed; position-indicating semaphore outside housing; noncorrosive working parts; pressure-greased fittings; pins, bolts, and nuts locked securely; state voltages of control circuit and heater supply.

Additional if Electrically Operated. Solenoid or motor; control relay to relieve duty on external control circuit; coils adequate for 15 successive operations without overheating; solenoid discharge resistor if solenoid-operated.

Additional if Pneumatically Operated. Motor-driven compressor; to charge from minimum to normal pressure in 30 minutes and from atmospheric to normal in 60 minutes; automatic pressure control to start compressor on pressure drop equivalent to three breaker operations and to stop at normal pressure; air-storage tank with capacity for five operations for breakers 115 kv and above, and eight operations 69 kv and below; safety valve; blowoff valve; shutoff valve; electrically operated control valve; air strainers; air cooler; pressure gage visible with housing doors closed; lock-out switch to prevent attempted operation with low air pressure; must not be possible for air leakage to build up pressure in closing cylinder; tee connection for attaching an emergency air supply; main switch and fuse or breaker for compressor motor; state voltage of compressor motor supply.

Accessories. Oil (see Chapter 41, Section 39); current transformer quantity, ratio, type, accuracy, name plates; painting inside and outside; tank lifter for frame-mounted breaker.

Spare Parts. Crosshead and lift rod for one phase; stationary contacts for one phase; interrupting chamber for one phase; trip coil; closing control relay coil; set of tank gaskets; bushing. For pneumatically operated breaker: coil for cut-off relay, electrically operated control valve, air-pressure-regulating switch, portable air-storage tank or hose to reach adjacent breaker, compressor, motor, and belt.

Factory Tests on Each Breaker. Tanks at 150 lb hydrostatic pressure for 30 min; mechanical, electrical, and pneumatic operations; manual close and trip; pressure test of entire air system, if pneumatically operated; high potential tests of current transformers and control circuits; dielectric test of breaker and bushings; impulse tests on one breaker.

Speed Tests on Each Breaker. Open at normal voltage; open at minimum voltage; close at normal voltage and pressure; close at normal voltage and minimum pressure; open and immediately reclose at normal voltage and pressure; repeat at normal voltage and minimum pressure; close and trip free at normal voltage and pressure; plot travel-cycle curves of above operations and mark: trip coil energized, arcing contacts part, breaker fully open (except for very high-speed breakers), breaker ready for reclosure, reclosing relay contacts close, arcing contacts touch, breaker fully closed.

Guaranteed Performance Data by Bidder. Continuous current capacity; interrupting current capacity CO + 15 sec + CO, ditto CO + 0 sec + CO; interrupting time with normal control voltage at 100 and at 25% of interrupting rating; ditto at less than 25%; reclosing time with normal control voltage and air pressure.

Design Data by Bidder. Type designation; exceptions to specification; momentary and 5-second ratings; test data to support guaranteed interrupting capacity; test facilities available; breaker tank inside diameter; material and thickness of tank wall; length of contact travel; number of breaks per phase; total break length; type of main and arcing contacts; method of extinguishing arc; minimum clearance and creepage of live parts to ground in oil and in air; type of closing mechanism; closing current; type of tripping mechanism; tripping current; bushing make, type, rating, weight, withstand voltages; type and operation of manual

closing device, tank vents; net weight with oil; maximum impact in terms of equivalent static weight; over-all dimensions; height to remove bushings; additional space for using manual closer; compressor motor rating; pressure settings and adjustment ranges; air-storage tank volume; number of stored operations; time for compressor to charge storage tank.

12. Switchboards. The switchboards of a hydroelectric generating plant constitute the centers of control, metering, and relaying of the main power circuits; control, protection, and distribution of energy from the control battery and chargers; control of the excitation circuits; and control and protection of the auxiliary-power circuits. In small stations these facilities are grouped on one or two boards centrally located, but in large stations the several functions are provided for on separate switchboards located at points about the station chosen to secure maximum convenience of operation and economy of wiring.

The main control, instrument, and relay boards control the main power equipment including the generators, the generator-voltage switching, the transformers, the transmission-voltage switching, and the several sources of auxiliary power supply. The diagram of Fig. 2 illustrates the controls, instrumentation, and relaying usually required. For a small station these facilities often are grouped on a single board near the generating unit or the governor. For an important station the main control boards are grouped in a control room centrally located with respect to the equipment controlled.

The control room should be well lighted, soundproofed, well ventilated, and preferably air-conditioned to provide the most favorable environment for this important operation.

Control rooms are variously arranged. In a large station the controls usually are located on a benchboard and the relays on a vertical board placed back to back with the instrument board. Graphic recorders may be located on the instrument or relay board or preferably on a separate vertical board. The battery charging and distribution controls generally are located in the control room to be directly under the supervision of the switchboard operators and are conveniently mounted on a separate vertical board. Figure 3 indicates a plan and cross-section of a modern control room in a large hydro station. For a small station with not more than two generating units, a compact and convenient arrangement can be obtained by placing the governor actuators in the control room. With the control room on the same floor with the base of the generator, and the two governor actuators in opposite ends of the control room, the piping from the actuators to the servomotors can be relatively short. By facing the two actuator boards toward each other and placing the controls, instruments, and relays on two back-to-back vertical boards, an arrangement can be obtained whereby one operator and an assistant can operate the entire station from a quiet, air-conditioned control room. These illustrations indicate the wide variety of control-room arrangements to be found.

The controls usually are arranged in partial conformity with the locations of equipment controlled, and mimic buses and connections are added so as to

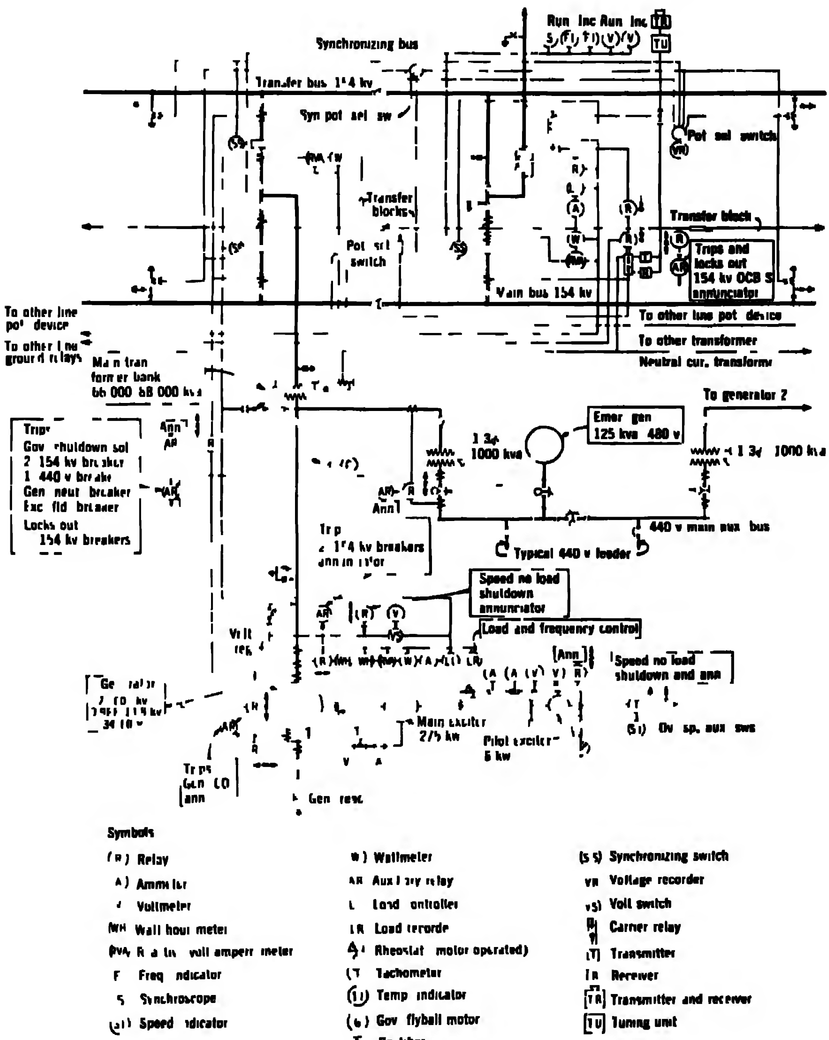
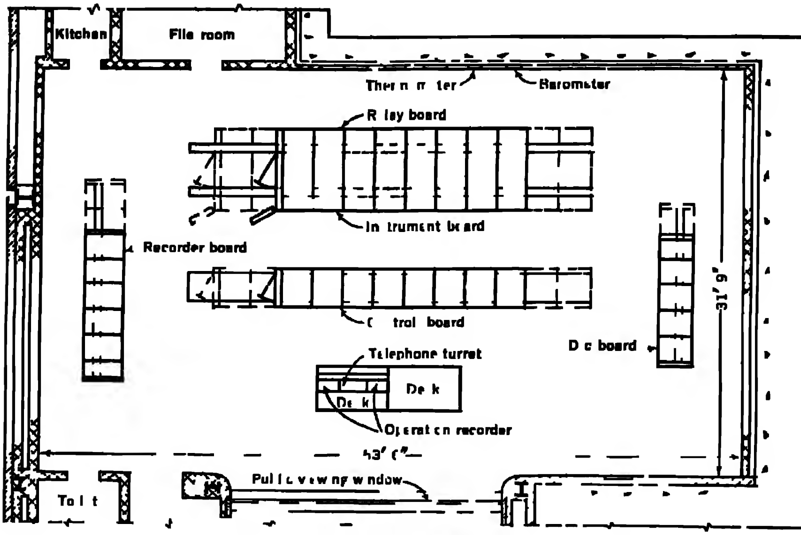
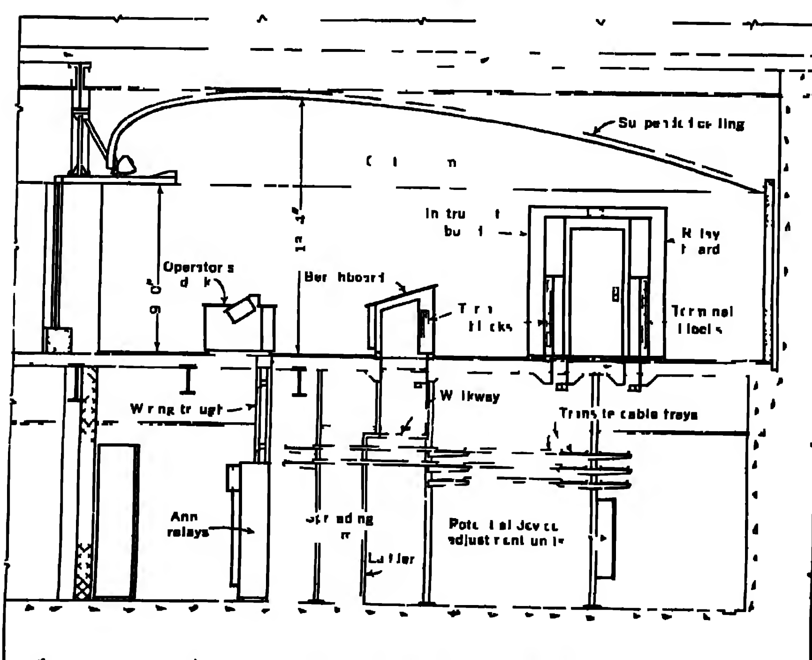


FIG 2 Typical main single line wiring diagram.

present on the face of the control board a physically arranged single-line diagram for the guidance of the operator Wiring from the switchboard devices is carried in readily accessible steel gutters to the floor below and distributed in conduits or preferably on open trays.



Plan of control room



Section through control room and spreading room

Fig 3 Typical control and spreading rooms of an important generating station

Excitation switching and control properly are kept to the minimum. Except where a central excitation scheme is used or where spare exciters are wired and switched, the excitation board consists merely of a single panel or cubicle for each generator mounted near the generator and controlled from the main control room. It contains the rheostats, field switches, regulator equipment, and associated devices. See Chapter 41, Section 26.

Auxiliary power, lighting, and heating switchboards usually are of the dead-front, steel-inclosed type, containing breakers for controlling and protecting the main circuits and fused switches and contactors for controlling and protecting the loads. In an important station, separate boards generally are used for control and protection of the various auxiliary power sources, distribution to the generating unit auxiliaries, distribution to the auxiliaries which are common to the entire station, and distribution to the lighting and heating systems. The sources of supply are often remotely controlled from the main control room, distribution is controlled manually at the distribution boards, and motors are controlled by push buttons and automatic devices at or near the equipment driven.

13. Instrument Transformers. Current and potential transformers are an essential part of the electrical equipment of every station to reduce the amperes and volts of the main circuits to values that are safe and economical for the operation of meters, relays, and other devices. This reduction must be effected at a substantially constant ratio and phase angle over the entire operating range. The accuracy is defined by A.I.E.E. Standards. The instrument transformers may be included in metal-clad switchgears, oil circuit-breakers, or switchboards, or may be purchased separately and installed in the station wiring.

Current transformers are inserted in series in the main circuits for the purpose of supplying secondary currents in proportion to the main currents. Standard current transformers at present have 5-ampere secondaries, although 1-ampere secondaries have advantages for long secondary leads and eventually may become standard. The insulation of these transformers must be equal to that of the primary circuit. Bushing-type current transformers are used in circuit-breaker and transformer bushings. Particular care must be taken in installing and operating current transformers to avoid excessive resistance in the secondary circuit or an open secondary circuit. In the former instance the load or burden will exceed the rating of the transformer and result in overheating and incorrect ratio; and in the latter instance high voltages will be produced in the secondary circuit which may damage equipment or be fatal if contacted.

Potential transformers are connected in shunt across the main circuits and produce secondary voltages in proportion to the main voltage. Standard potential transformers have 115- or 120-volt secondaries. Standard sizes are rated 50, 100, 500, and 1000 volt-amperes. Excessive load or low-power-factor load will result in ratio error. Potential transformers usually are connected to the primary circuits through potential fuses and current-limiting resistors.

Several structural forms and mounting arrangements are available for current and potential transformers to suit wiring conditions both for outdoor and for indoor service. The secondaries usually are wired through test blocks and switches so that the current transformers may be short-circuited and the potential transformers open-circuited when work is to be done on the secondary circuits. The secondaries always should be grounded, preferably close to the transformers.

14. Generator-voltage Wiring. The main connections from the generators through the switchgear to the transformers, including the generator buses, if any, require careful design on account of the heavy currents which result in heating and mechanical stresses. Two general types of wiring are common: (1) bare bars, tubes, and shapes on insulators; and (2) insulated cables on insulators or in ducts.

Bare conductors are preferred for short connections, particularly in housings and cubicles where many connections and taps are made to breakers, disconnects, instrument transformers, and other apparatus. They often are preferred also for long runs in tunnels or housings for heavy currents. Bare conductors of copper and aluminum are available in flat bars, tubes, angles, channels, and hollow squares, and in various dimensions and weights. Comparative characteristics of the two bus-bar metals are indicated by the following:

	Copper	Aluminum
Weight of 1 sq in. cross-section in pounds per foot of length	3.864	1 172
D-c resistance of 1 sq in. cross-section in ohms per foot at 20° C	0.00000831	0.00001336
Average linear coefficient of expansion per centigrade degree between 25° C and 100° C	0.0000168	0.000023

For d-c circuits and light a-c circuits, flat bars are preferred because of the simplicity of the bolting or clamping. For heavy a-c circuits, hollow shapes are preferred on account of skin effect. Ventilation always is desired to help dissipate heat losses. Such shapes as hollow squares with holes for ventilation, and hollow squares formed of two channels or two angles with legs toward each other but separated to allow good ventilation, are most economical for heavy a-c circuits.

Operating temperature is standardized by A.I.E.E. at not over 70° C when copper-to-copper bolted or clamped connections are used, and not over 85° C when silver-plated, brazed, or welded connections are used. As a very general indication of the amount of copper required, four ¼- by 4-in. copper bars standing on edge with ¼-in. vertical ventilation between bars and with plain bolted or clamped connections will carry about 2750 amperes at 60 cycles with a 30° C rise above 40° C in still, open air. If the bars are inclosed in a cubicle or housing, the temperature will be increased about 15 Centigrade

degrees and the joints should be brazed or welded or may be silver-plated and bolted. The same weight of copper used in two copper channels arranged to form a hollow square with vertical ventilation would carry about 3820 amperes, and if used in two angles arranged to form a hollow square with vertical ventilation it would carry about 4870 amperes. The economical design of a circuit of this kind involves material and shape of conductor, type of joints, skin effect, proximity effect, type of supports, mechanical stresses from short circuit and from thermal expansion, type of inclosure, amount of natural or forced ventilation, and ambient temperatures. Reference should be made to the abundant material found in copper and aluminum manufacturer's data books and the various electrical engineers' handbooks (see Chapter 40, Section 17, Refs. 9, 10, 11, 12, 18, 20, and 22).

Bus supports usually are specified to withstand a wet flashover test of 2½ times working voltage between phases, and a minimum mechanical breaking strength of two or three times the maximum computed short-circuit stress with load applied in any direction through the point of intersection of the center line of the conductor and the axis of the support (see Chapter 40, Section 13) Porcelain is considerably stronger under compression than under any other loading; and where short-circuit stresses are very heavy this advantage sometimes is utilized by surrounding the bus with two, three, or four porcelain insulators at each support so that all stresses are applied in compression.

Structures and inclosures for buses, breakers, disconnects, instrument transformers, and related equipment are built of concrete, masonry, Transite, steel, copper, aluminum, or combinations of these materials. Metal structures and inclosures are now used very extensively. They can be fabricated in the shop in fairly large sections and quickly set in place in the plant. Sometimes the equipment and housing are purchased from one manufacturer and completely assembled before shipment, but many power-plant builders prefer to accumulate the equipment and install it after the housing is in place in the plant. Steel housings are used considerably for circuits carrying moderate currents except that any barriers between phases and any single-phase sections should be of nonmagnetic material. Aluminum has the advantage of a permanent finish which does not require painting, light weight, easy fabrication, and, according to some authorities, good resistance to burning from arcs. The structures should be of ample size to allow not only for necessary clearances, but also for inspection and cleaning.

Reinforcing steel arranged in loops around a single phase of an a-c circuit carrying more than about 2000 amperes should be avoided, as such steel will become heated and crack the concrete. Steel beams and reinforcing, even though not looped around a single phase, should be kept at least 12 in. away from conductors in the 15-kv class and should be thoroughly grounded.

Insulated cables generally are used for long uninterrupted runs from generator switchgears to transformers. They have the advantages of having continuous copper without bolted joints and of being more compact than bare

conductors in housings. Cables insulated with ozone-resisting rubber, varnished cambric, or oil-impregnated paper, and with proper outer coverings, have been used for these circuits. Three-conductor cables can be used for small-capacity circuits, but above 300 to 400 amperes per phase single-conductor cables are necessary. Under usual station conditions it is not possible to carry more than about 1000 amperes on a single cable, so that for heavy

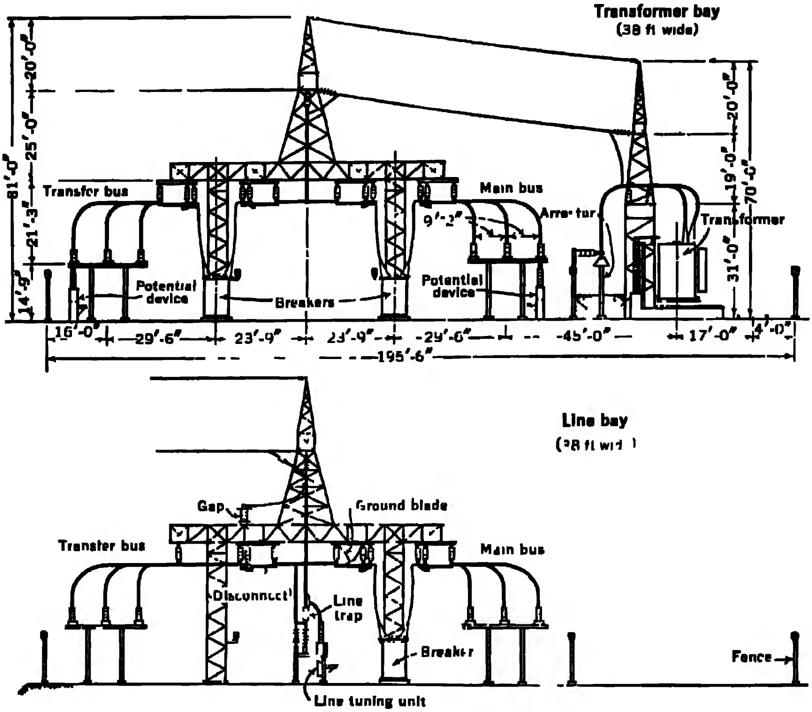


FIG. 4. Typical rigid-type switchyard construction for level terrain (154 kv).

cable circuits it usually is necessary to use two or three cables per phase. When several cables are used in a bank, they seriously react on one another and will not share the load evenly unless the configuration is worked out carefully to reduce proximity effects [3].

A development of about 1940 in heavy-capacity conductors for power-station use consists of paper-insulated, shielded, D-armored cables with all three phases installed in a continuous steel pipe filled with oil maintained under pressure. Some economy in copper is gained by virtue of the efficient heat transfer from the conductors through the insulation and the oil to the outer surface of the pipe. A considerable saving in space is gained over that required by bare conductors in housings or that required by insulated cables in duct lines [10]. Another development consists of bare conductors of square

cross-section with each phase installed on insulators within a cylindrical, nonmagnetic metal housing. Four porcelain post-type insulators are used at each support so that all porcelain is in compression. The separate grounded housing around each phase insures that all faults will be phase-to-ground [12].

15. Transmission-voltage Wiring. With the high-voltage wiring, the problem is not one of current-carrying capacity and short-circuit stresses, as

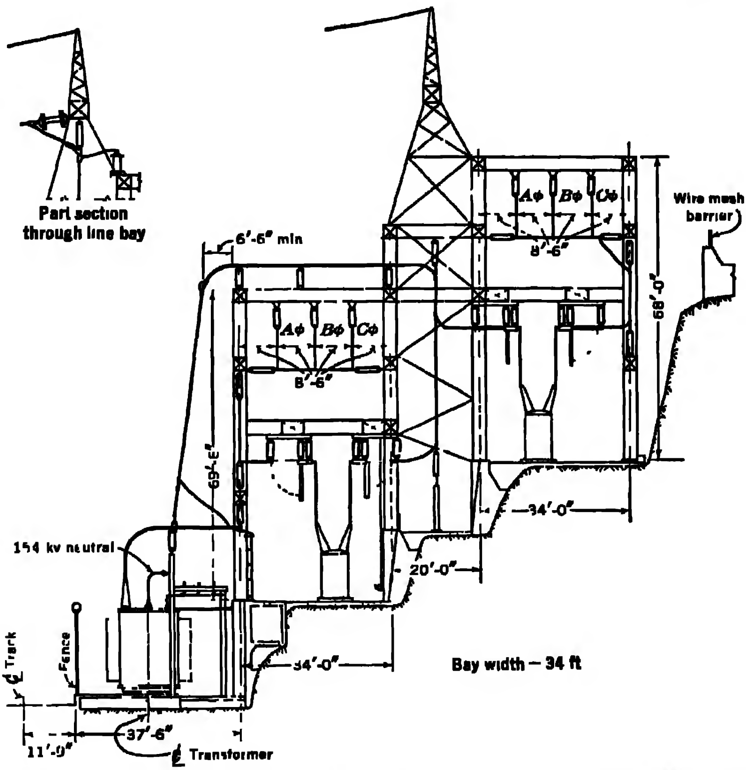


FIG. 5. Adaptation of rigid-type switchyard construction for hillside terrain (154 kv).

with the low-voltage circuits, but principally of insulation. Large clearances must be maintained, and usually the entire installation is out-of-doors. Flexibility is required, so that structures can be extended and circuits added with minimum shutdown. (See Section 1.)

Two general types of construction have been developed: (1) the flexible type, where buses and connections are of stranded cable and are suspended by strain insulators between steel structures; and (2) the rigid type, where buses and most of the connections are of pipe or tubing and are supported by post insulators above or below steel structures. The second type seems to be

gaining favor because less spacing between conductors can be allowed with the rigidly supported conductors and usually the steel structures can be lighter and more compact. A combination of the two types may be desirable.

An example of the rigid-type construction is given in Fig. 4. Buses and main connections are of 4-in. and 2½-in. steel pipe with joints welded and zinc-sprayed. Recommended clearances and spacings for various voltages are given in Table 1. For current-carrying capacities of bare cables and of steel pipes, see Section 28, Refs. 1, 7, and 13.

TABLE 1

RECOMMENDED MINIMUM CLEARANCES AND SPACINGS FOR OUTDOOR EQUIPMENT AND RIGID CONDUCTORS

Voltage between Phases, kv	Clearance, Live Part to Ground, in.	Clearance, Live Part to Live Part, in.	Spacing, Phase to Phase, in.
15	9	12	24
23	13	18	30
34.5	16	22	36
46	20	27	42
69	28	38	56
115	44	59	84
138	53	72	94
161	61	82	106
230	86	115	144

An adaptation of the rigid construction for hillside topography as often found at a tributary hydro plant is shown in Fig. 5. The same general arrangement is also suitable on level ground if the site is too narrow to permit the flat structures of Fig. 4.

For outdoor structures, refer to Chapter 43, Section 29.

16. Auxiliary Power Wiring. The wiring from the auxiliary power distribution boards to the motors, controls, heaters, and lighting devices generally is installed in fiber and steel conduits concealed in the powerhouse structures. Varnished-cambric, rubber, and rubber-like insulations are suitable, rubber-like insulations being used most generally. Outside coverings should be of Neoprene or other moisture- and oil-resisting material. Three-conductor cables can be used for motor circuits, but some builders prefer all single-conductor to avoid waste in cutting lengths. Control wires from the push buttons at the motors sometimes are run back to the contactors on the distribution board through the same conduits with the power wires.

For lighting and small heating circuits operating at not above 600 volts, thermoplastic compounds are available which provide the insulation and covering in one homogeneous wall. The material can be colored as desired for coding.

17. Control Wiring. Wiring from the control switchboards to breakers, instrument transformers, signal devices, and other equipment within the station and in the switchyard usually is in the form of multiple-conductor cables. A separate cable usually is run for each circuit, such as the control of a breaker or a rheostat, or the leads from a set of instrument transformers. Stranded conductors are preferable to avoid breakage from vibration. Oil-base insulating compounds using synthetic rubber are widely employed at present. They resist moisture, oil, and ozone and have long life. Thin coverings of Neoprene often are applied over the insulation of each conductor, and a tough Neoprene jacket is placed over the group to form the cable. Though many kinds of insulation, covering, and cable make-up are satisfactory, the general type just described is very suitable either indoors or outdoors.

On the switchboard, the varnished-cambric, asbestos panel wiring is brought to terminal blocks that connect it to the station cables. It is customary to tag every terminal stud, wire, and cable for identification. From the terminal blocks, the control cables are routed throughout the station to the various equipments controlled. A studied arrangement of this important wiring is essential to secure an orderly, flexible arrangement. In a small station it is quite possible to carry the cables in individual steel conduits to their various destinations, but in a medium or large station a much more systematic arrangement is necessary. Many designers of large stations provide a spreading room directly below the control room and a cable gallery that runs the entire length of the powerhouse. The control cables are carried from the switchboard terminal blocks through the floor to the spreading room where they fall systematically onto cable trays that extend along the walls of the cable gallery to points opposite the generators, switchgears, and other equipment. The cables then leave the trays and are carried in comparatively short steel conduits to their destinations. A similar gallery or a duct and manhole system is used to carry the cables to the equipment in the switchyard. Such an arrangement is compact, and yet each cable is accessible for most of its length. Cable trays of Transite with bottom and sides formed of one piece in about 13-ft lengths are available.

18. Wires and Cables. Conductors for power-station cables are of soft-drawn annealed copper. Tables 2 and 3 give data on solid and concentrically stranded conductors. Solid conductors often are used in size 10 and smaller and stranded in size 8 and larger, but it often is advisable to use stranded conductors in smaller sizes for important controls and for small motors, especially where the wiring is subjected to vibration or frequent bending. For many years control cables have been made with flexible 19-strand conductors even in the small sizes, but standard 7-strand conductors are coming into favor. Flexible stranding is generally used in sizes larger than 500,000 cir mils when the cables are to be pulled into ducts. For 3-conductor cables, the conductors usually are sector-shaped to reduce the over-all diameter of the cable. In some cases the sector-shaped conductors are crushed in rolls to form a compact stranding which further reduces the over-all diameter of the

TABLE 2 *

WORKING TABLE, STANDARD ANNEALED COPPER WIRE

American Wire Gage (B. & S.)

Gage No.	Diameter in Mils	Cross-section: Circular Mils	Cross-section: Square Inches	Ohms per 1000 Ft: 25° C (=77° F)	Ohms per 1000 Ft: 65° C (=149° F)	Pounds per 1000 ft
0000	460	212,000	0.166	0.0500	0.0577	641
000	410	168,000	0.132	0 0630	0.0727	508
00	365	133,000	0.105	0.0795	0.0917	403
0	325	106,000	0.0829	0.100	0.116	319
1	289	83,700	0.0657	0 126	0.146	253
2	258	66,400	0.0521	0.159	0.184	201
3	229	52,600	0.0413	0.201	0.232	159
4	204	41,700	0.0328	0.253	0.292	126
5	182	33,100	0.0260	0.319	0.369	100
6	162	26,300	0.0206	0.403	0.465	79.5
7	144	20,800	0.0161	0.508	0 586	63.0
8	128	16,500	0.0130	0.641	0.739	50.0
9	114	13,100	0 0103	0.808	0.932	39.6
10	102	10,400	0.00815	1 02	1.18	31.1
11	91	8,230	0.00647	1.28	1.48	24.9
12	81	6,530	0.00513	1 62	1.87	19 8
13	72	5,180	0.00407	2.04	2.36	15.7
14	64	4,110	0.00323	2.58	2.97	12.4

* From *Bureau of Standards Circular* 31.

cable. Such cables, while smaller, are stiffer and more difficult to splice. For single-conductor cables for a-c circuits, the conductor often is stranded with a hollow core which reduces the skin effect and permits more current per pound of copper but increases the over-all diameter. Another method of reducing the skin effect without increasing the diameter appreciably is to divide the conductor into three or four equal groups separated by a small amount of radial insulation. This type of conductor, for heavy single-phase cables, is called the segmental conductor. The strands are sometimes partially crushed to give smaller cable diameter.

Insulations of three general types are in common use. Oil-base rubber insulation resists moisture, oil, and ozone. It is fairly soft, however, and has a slight tendency to become eccentric after years of service at rated temperature, especially if used on a heavy conductor. Synthetic rubbers have been developed with electrical and mechanical characteristics almost as good as those of natural rubber; some are better. Varnished-cambric insulation re-

TABLE 3 *

Bare Concentric-lay Cables of Standard Annealed Copper

Size of Cable		Ohms per 1,000 Ft		Pounds per 1,000 Ft	Standard Concentric Stranding			Flexible Concentric Stranding		
Circular Mils	A.W.G. No.	25° C (=77° F)	65° C (=149° F)		Number of Wires	Diameter of Wires, mils	Outside Diameter, mils	Number of Wires	Diameter of Wires, mils	Outside Diameter, mils
2,000,000	...	0.00539	0.00622	6,180	127	125.5	1,631	169	108.8	1,632
1,900,000	...	.00568	.00655	5,870	127	122.3	1,590	169	106.0	1,590
1,800,000	....	.00599	.00692	5,560	127	119.1	1,548	169	103.2	1,548
1,700,000	....	.00634	.00732	5,250	127	115.7	1,504	169	100.3	1,504
1,600,000	....	.00674	.00778	4,940	127	112.2	1,459	169	97.3	1,460
1,500,000	....	.00719	.00830	4,630	91	128.4	1,412	127	108.7	1,413
1,400,000	...	.00770	.00889	4,320	91	124.0	1,364	127	105.0	1,365
1,300,000	....	.00830	.00958	4,010	91	119.5	1,315	127	101.2	1,315
1,200,000	...	.00899	.0104	3,710	91	114.8	1,263	127	97.2	1,264
1,100,000	...	.00981	.0114	3,400	91	109.9	1,209	127	93.1	1,210
1,000,000	...	.0108	.0124	3,090	61	128.0	1,152	91	104.8	1,153
950,000	...	.0114	.0131	2,930	61	124.8	1,123	91	102.2	1,124
900,000	...	.0120	.0138	2,780	61	121.5	1,093	91	99.4	1,094
850,000	....	.0127	.0146	2,620	61	118.0	1,062	91	96.6	1,063
800,000	....	.0135	.0156	2,470	61	111.5	1,031	91	93.8	1,031
750,000	...	.0144	.0166	2,320	61	110.9	998	91	90.8	999
700,000	....	.0154	.0178	2,160	61	107.1	964	91	87.7	965
650,000	...	.0166	.0192	2,010	61	103.2	929	91	84.5	930
600,000	....	.0180	.0207	1,850	61	99.2	893	91	81.2	893
550,000	...	.0196	.0226	1,700	61	95.0	855	91	77.7	855
500,000	....	.0216	.0249	1,540	37	116.2	814	61	90.5	815
450,000	...	.0240	.0277	1,390	37	110.3	772	61	85.9	773
400,000	...	.0270	.0311	1,240	37	104.0	728	61	81.0	729
350,000	...	.0308	.0356	1,080	37	97.3	681	61	75.7	682
300,000	....	.0360	.0415	926	37	90.0	630	61	70.1	631
250,000	...	.0431	.0498	772	37	82.2	575	61	64.0	576
212,000	0000	.0509	.0587	653	19	105.5	528	37	75.6	533
168,000	000	.0642	.0741	518	19	94.0	470	37	67.3	471
133,000	00	.0811	.0936	411	19	83.7	418	37	60.0	420
106,000	0	.102	.117	326	19	74.5	373	37	53.4	374
83,700	1	.129	.149	258	19	66.4	332	37	47.6	333
66,400	2	.162	.187	205	7	97.4	292	19	59.1	296
52,600	3	.205	.237	163	7	86.7	260	19	52.6	263
41,700	4	.259	.299	129	7	77.2	232	19	46.9	234
33,100	5	.326	.376	102	7	68.8	206	19	41.7	209
26,300	6	.410	.473	81.0	7	61.2	184	19	37.2	186
20,800	7	.519	.599	64.3	7	54.5	164	19	33.1	166
16,500	8	.654	.755	51.0	7	48.6	146	19	29.5	147

* From *Bureau of Standards Circular* 31.

Note: This table is in accord with standards adopted by the Standards Committee of the American Institute of Electrical Engineers, both in respect to the number of wires and in respect to the correction for increase of resistance and mass due to the twist of the wires. The values given for ohms per 1000 ft and pounds per 1000 ft are 2% greater than for a solid rod of cross-section equal to the total cross-section of the wires of the cable.

sists moisture, oil, and ozone. It cannot become appreciably eccentric because it consists of tapes wrapped to required thickness. Where varnished-cambric cables are used in vertical runs, they should be built with nonmigrating compound to prevent leakage at the lower ends. Oil-impregnated paper insulation is compact and least expensive. It does not resist moisture and must therefore be covered by a continuous lead sheath and sealed at the ends. The insulation is wrapped and does not become eccentric.

Radial thicknesses of insulation in sixty-fourths of an inch and safe operating conductor temperatures in degrees Centigrade for single-conductor cables of 1,000,000 cir mil size and larger are as follows for operation in an ungrounded circuit at the phase-to-phase voltages stated:

	5 kv	10 kv	15 kv
Insulation thickness			
Rubber	12	19	28
Cambric	10	15	21
Paper	7	12	16
Conductor temperature			
Rubber	75	70	70
Cambric	85	81	77
Paper	85	83	81

Shielding in the form of thin metallic tapes or conducting impregnating braids is required over the insulation of all cables operating at voltages above 5000 volts between conductors, to distribute the voltage stresses uniformly and avoid formation of corona and ozone. These shields should be grounded and must be belled out at the ends to form voltage stress cones.

Coverings for dry locations consist of tapes and braids The braids usually are impregnated to resist moisture and flame. For wet locations, lead sheaths or rubber jackets sometimes reinforced with fabric are used. Lead sheaths often deteriorate in alkali environment and usually must be protected by waterproof tapes. Jackets of synthetic rubber such as Neoprene are highly resistive to moisture, oil, and abrasion.

The grounding of shielding tapes and lead sheaths must be done judiciously. If the conductor current is high, it is preferable to ground only at one point so as to avoid excessive sheath currents If the cable is long, it is preferable to ground frequently and usually necessary to sectionalize the sheath to avoid excessive sheath voltages. For an extension of this subject, refer to Section 28, Ref. 11, and to the various electrical engineers' handbooks.

Splices and terminals must be insulated carefully, using materials matching the cable insulation. The success of the splice or terminal depends in a large measure on the skill of the splicer For important cables, instructions as to materials and workmanship should be obtained from the cable manufacturer. Rubber-insulated cables can be terminated by removing the outer covering and shielding tapes for a sufficient distance back from the end to avoid creepage current from the conductor to ground. In dry locations varnished-cam-

TABLE 4

Recommended Current-carrying Capacities of Insulated Conductors in Conduits

Based on single-conductor wires and cables, 600-volt insulation, continuous operation, not more than 3 conductors per conduit, ambient temperature 30° C

Size, AWG or Mcm	Type and Carrying Capacity in Amperes					
	R RW 60° C	RH V 75° C	P AVB 85° C	AVA 110° C	AIA 125° C	A 200° C
14	15	15	25	30	30	30
12	20	20	30	35	40	40
10	30	30	40	45	50	55
8	40	45	50	60	65	70
6	55	65	70	80	85	95
4	70	85	90	105	115	120
3	80	100	105	120	130	145
2	95	115	120	135	145	160
1	110	130	140	160	170	190
1/0	125	150	155	190	200	225
2/0	145	175	185	215	230	250
3/0	165	200	210	245	265	285
4/0	195	230	235	275	310	340
250	215	255	270	315	335	
300	240	285	300	345	380	
350	260	310	325	390	420	
400	280	335	300	420	450	
500	320	380	405	470	500	
600	355	420	455	525	545	
700	385	460	490	560	600	
750	400	475	500	580	620	
800	410	490	515	600	640	
900	435	520	555			
1000	455	545	585	680	730	
1250	495	590	645			
1500	520	625	700	785		
1750	545	650	735	...		
2000	560	665	775	840		

TABLE 4 —*Continued*

	Correction Factors for Ambients over 30° C					
Ambient °C	R RW 60° C	RH V 75° C	P AVB 85° C	AVA 110° C	AIA 125° C	A 200° C
40	0.82	0.88	0.90	0.94	0.95	. . .
45	.71	.82	.85	.90	.92	
50	.58	.75	.80	.87	.89	
55	.41	.67	.74	.83	.86	. . .
60		.58	.67	.79	.83	0.91
70		.35	.52	.71	.76	.87
75			.43	.66	.72	.86
80			.30	.61	.69	.84
90				.50	.61	.80
100	.				.51	.77
120				.		.69
140		. .				.59

Correction Factors for Operating Conditions Other Than Those Specified

Working Voltage	% Amperes
600 volts	100
5,000 volts	95
15,000 volts	90

Duration of Operation	% Amperes
Continuous	100
2 hr	105
1 hr	125

Number of Loaded Conductors per Conduit	% Amperes
1 to 3	100
4 to 6	80
7 to 9	70

' Type Letters and Insulations.

A = asbestos.
AIA = impregnated asbestos, asbestos braid.
AVA = asbestos, varnished cambric, asbestos braid.
AVB = asbestos, varnished cambric, flame-resistant cotton braid.
P = paper, oil impregnated.
R = code grade rubber.
RH = heat-resistant rubber.
RW = moisture-resistant rubber.
V = varnished cambric.

bric-insulated cables can be terminated like rubber insulation except that the exposed insulation should be covered thoroughly with a waterproof varnish. For damp locations a compound-filled cable terminal with porcelain bushings should be used. Paper-insulated cables require compound-filled cable terminals whether indoors or outdoors.

Current-carrying capacities of wires and cables, where voltage drop is not a determining factor, are determined by the temperature considered safe for the insulation used. The operating temperature depends on many conditions, such as conductor type, insulation thickness, number of conductors per cable, number of cables per duct, number of ducts per duct bank, ambient temperature, and load factor. Approximate current-carrying capacities of wires and cables in conduits and ducts, with commonly used insulations, for voltages up to 15,000 volts, and under usual operating conditions are given in Table 4. For important cables, reference should be made to Section 28, Ref. 9.

Dimensions and weights of insulated wires and cables commonly used in powerhouse construction are given in Tables 5, 6, 7, and 8. The data are

TABLE 5

APPROXIMATE DIAMETERS AND WEIGHTS OF SINGLE-CONDUCTOR, CONCENTRIC-STRAND, RUBBER-INSULATED, RUBBER-JACKETED POWER CABLES FOR UNGROUNDED SERVICE

Conductor Size, A.W.G. or Mcm	Outside Diameter, in.			Net Weight, lb per 1000 ft		
	600-v	3000-v	15,000-v	600-v	3000-v	15,000-v
12	0.35		....	70	....	....
10	0.37		..	90	...	....
8	0.41	0.63		120	200	....
6	0.48	0 68	.	175	260	....
4	0.54	0.73		235	330	....
2	0.61	0 79	..	310	430	....
1	0.68	0.81		420	530	.
1 0	0 72	0.87	1 60	500	650	1310
2 '0	0.77	0.93	1.65	600	750	1450
3/0	0.82	1.02	1.70	750	900	1600
4 '0	0.90	1.08	1.75	915	1050	1790
250	1.01	1.13	1.80	1100	1250	1950
300	1.03	1.18	1.85	1250	1400	2170
350	1.08	1.23	1.90	1450	1600	2380
400	1.13	1.29	1.95	1650	1800	2620
500	1.22	1.40	2.05	1950	2100	2970
600	1.36	1.48	2.15	2300	2500	3370
700	1.43	1.56	2 25	2600	2900	3730
750	1.46	1.59	2.30	2850	3100	3920
800	1.50	....	2.35	3100	....	4110

TABLE 6

Approximate Diameters and Weights of Single-conductor, Concentric-strand Varnished-cambric-insulated, Braided Power Cables for Ungrounded Service

Conductor Size, A.W.G. or Mcm	Outside Diameter, in.			Net Weight, lb per 1000 ft		
	1000-v	5000-v	15,000-v	1000-v	5000-v	15,000-v
6	0.37	0.50	0.75	130	180	290
4	0.42	0.55	0.80	180	240	360
2	0.48	0.61	0.86	270	320	460
1	0.55	0.65	0.92	340	390	560
1/0	0.59	0.69	0.96	420	470	640
2/0	0.64	0.73	1.00	510	560	750
3/0	0.69	0.79	1.06	620	670	880
4/0	0.75	0.84	1.14	770	820	1070
250	0.83	0.94	1.19	910	990	1200
300	0.90	1.00	1.25	1090	1170	1400
350	0.96	1.05	1.30	1260	1340	1580
400	1.00	1.12	1.34	1420	1530	1760
500	1.12	1.21	1.43	1780	1870	2110
600	1.23	1.29	1.51	2140	2230	2460
750	1.33	1.40	1.61	2660	2710	2990
1000	1.49	1.55	1.77	3460	3540	3830
1250	1.69	1.73	1.94	4360	4400	4710
1500	1.81	1.84	2.06	5170	5210	5550
1750	1.92	1.95	2.17	5950	6000	6370
2000	2.01	2.04	2.26	6810	6870	7260

approximate but are believed to be maximum and sufficiently accurate for estimating and preliminary layout.

19. Conduits and Ducts. Rigid iron conduit is used to inclose the greater part of the station wiring. It is manufactured from steel tubing and either enameled, sherardized, electrogalvanized, or hot-galvanized. The interior, and in some types the exterior also, is enameled over the zinc coating. It is made in 10-ft lengths, reamed and threaded at each end, and each length is provided with one coupling. Weights and dimensions of conduit and principal fittings are given in Table 9. It should be noted that dimension "U" is the radius of a standard factory bend. A field bend usually is made at about double the standard radius.

Aluminum and brass conduits of about the same dimensions as standard iron conduits are available for cases where it is necessary to run each phase of an a-c circuit in a separate conduit. Flexible iron and brass conduits are available for connections to motors and machines where some movement is desired.

TABLE 7

APPROXIMATE DIAMETERS AND WEIGHTS OF SINGLE-CONDUCTOR, CONCENTRIC-STRAND, PAPER-INSULATED, LEAD-COVERED POWER CABLES FOR UNGROUNDED SERVICE

Conductor Size, A.W.G. or Mcm	Outside Diameter, in.			Net Weight, lb per 1000 ft		
	1000-v	5000-v	15,000-v	1000-v	5000-v	15,000-v
4	0.54	0.61	0.93	690	790	1470
2	0.60	0.67	0.99	850	960	1660
1	0.61	0.67	1.00	910	1020	1700
1/0	0.65	0.71	1.04	1030	1140	1840
2/0	0.69	0.74	1.10	1170	1280	2140
3/0	0.73	0.81	1.14	1340	1560	2350
4/0	0.80	0.86	1.19	1650	1760	2590
250	0.85	0.91	1.24	1810	1930	2790
300	0.90	0.96	1.29	2030	2170	3040
350	0.94	1.01	1.33	2260	2390	3290
400	0.99	1.05	1.38	2470	2500	3530
500	1.06	1.14	1.47	2880	3180	4150
750	1.28	1.31	1.64	4130	4200	5270
1000	1.44	1.47	1.80	5280	5370	6550

TABLE 8

APPROXIMATE DIAMETERS AND WEIGHTS OF RUBBER-INSULATED, RUBBER-JACKETED,* 600-VOLT, 7-STRAND, CONTROL CABLES

Number of Conductors	Outside Diameter, in.		Net Weight, lb per 1000 ft	
	No. 12	No. 9	No. 12	No. 9
1	0.25	0.27	45	70
2	0.60	0.67	160	280
3	0.67	0.75	260	340
4	0.72	0.81	290	420
5	0.82	0.88	340	520
6	0.85	0.95	410	590
7	0.85	0.95	450	600
8	0.92	1.05	550	750
9	1.01	1.15	630	840
10	1.08	1.19	650	930
11	1.10	1.21	700	1020
12	1.15	1.26	730	1110
14	1.17	1.32	830	1200
19	1.25	1.48	1050	1500

* A thin jacket over the insulation of each conductor and an outer jacket over-all.

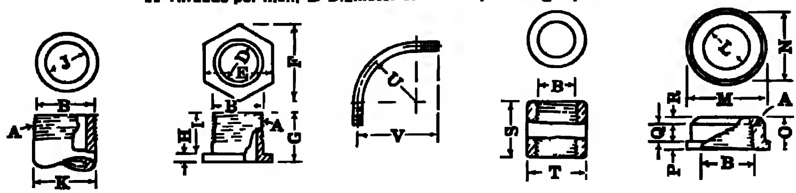
TABLE 9

WEIGHTS AND DIMENSIONS: RIGID IRON CONDUIT AND FITTINGS

Dimensions—Inches

Conduit Nipples Elbows Couplings Bushings

A-Threads per inch; B-Diameter of threads; C-Weight per ft in lb



Size of Conduit, in	A	B	C	D	E	F	G	H	I	J	K
½	14.0	0.82	0.85	0.02	1 00	1.15	0.62	0.12	0.50	0.62	0.84
¾	11.0	1.02	1.13	0 82	1.25	1.44	0.81	0.10	0.62	0.82	1.05
1	11 5	1.28	1.68	1.01	1.37	1.39	0.94	0.25	0 69	1.04	1.31
1¼	11.5	1.63	2.28	1.38	1.75	2.02	1.06	0.25	0.81	1.38	1 63
1½	11 5	1.87	2.73	1.61	2.00	2.31	1 12	0.31	0 81	1.61	1.90
2	11.5	2.34	3 68	2.06	2.50	2.89	1.31	0 31	1.00	2 06	2.37
2½	8.0	2.82	5.82	2.46	3 00	3 46	1.44	0 37	1.06	2.46	2.87
3	8.0	3.41	7 62	3 06	3.75	4.33	1.50	0.37	1.12	3 06	3 50
3½	8.0	3 94	9.20	3.34	4.25	4.91	1.62	0.44	1.19	3.51	4 00

Size of Conduit, in.	L	M	N	O	P	Q	R	S	T	U	V
½	0 62	1.00	0 94	0 37	0.12	0.19	0.06	1.37	1.12	4.25	7 38
¾	0.75	1 25	1 12	0 44	0 12	0.25	0.06	1 56	1 31	5.38	8 38
1	1.00	1.50	1.37	0.50	0 16	0.25	0.09	1 75	1.62	5 75	9 50
1¼	1.25	1.81	1 75	0 56	0.19	0 28	0.09	2 12	2 00	7 25	10 88
1½	1.50	2.12	2 00	0.56	0 19	0 28	0.09	2.50	2 25	8.25	12 63
2	1.94	2 56	2 37	0 62	0 19	0 31	0.12	2 62	2.75	9.50	15.25
2½	2.37	3 06	2 87	0 75	0 25	0 37	0.12	2 87	3 31	10.50	17.38
3	2.87	3 75	3 50	0 81	0 25	0 37	0.19	3 06	3.93	13 00	19.50
3½	3.25	4 25	4.00	1.00	0.37	0.44	0.19	3.62	4 43	15 00	21 25

Weight - Pounds

Size of Conduit, in.	Quantity in Feet						
	500	1,000	2,000	3,000	4,000	5,000	6,000
½	426	852	1,704	2 556	3,408	4,200	5,112
¾	567	1 134	2 268	3,402	4 536	5 670	6,804
1	812	1 681	3 368	5,052	6 736	8 420	10 104
1¼	1,141	2,281	4 562	6 843	9 124	11 405	13 686
1½	1,366	2,731	5 462	8 193	10 924	13,655	16 386
2	1 839	3 678	7 356	11,034	14 712	18 390	22 068
2½	2,910	5 819	11 638	17 457	23 276	29 095	34 914
3	3 808	7 616	15 232	22 848	30 464	38,080	45 696
3½	4 601	9 202	18,404	27,606	36,808	46,010	55 212

Size of Conduit, in.	Quantity in Feet						
	7,000	8,000	9,000	10,000	15,000	20,000	25,000
½	5,964	6,816	7 668	8 520	12,780	17,040	21 300
¾	7,938	9,072	10 206	11 340	17 010	22,680	28 350
1	11,788	13,472	15,156	16 840	25,260	33,680	42,100
1¼	15,967	18,248	20,529	22,810	34,215	45 620	57,025
1½	19,117	21,848	24 579	27,310	40,965	54,620	68,275
2	25,746	29 424	33 102	36 780	55 170	73 560	91 950
2½	40 733	46,552	52,371	58 190	87,285	116 380	145,475
3	53 312	60 928	68 544	76,160	114,240	152,320	190,400
3½	64,414	73,616	82,918	92,020	138 030	184,040	230,050

Fiber and Transite conduits are available for duct lines and for station use. They are lighter and less expensive than iron conduit, and, being nonmagnetic, they are suitable for running individual phases in individual conduits. Fiber conduit is made of felted fibers and coal-tar pitch; Transite is made of asbestos fibers and Portland cement. Both are made in light weights for inclosing in concrete and in heavier weights for running exposed or laying in the earth. Couplings of the Harrington type are available to fit the tapered ends of the conduit lengths. Weights and dimensions are given in Tables 10 and 11.

TABLE 10

WEIGHTS AND DIMENSIONS OF FIBER CONDUIT *

Nominal Inside Diameter, in.	Approximate Outside Diameter, in.	Net Weight, lb per foot	Radius of 45° and 90° Bend, in.	Radius of 45° and 90° Elbow, in.	Standard Length, ft
		Standard Weight			
1	1.41	0.60	18, 24, 36	5.75	5
1½	2.10	0.85	18, 24, 36	8.25	5
2	2.56	1.05	18, 24, 36	9.5	5
2½	3.06	1.30	24, 36	10.5	8
3	3.50	1.40	36	13.0	8
3½	4.02	1.60	36	15.0	8
4	4.50	1.90	36	16.0	8
4½	5.10	2.20	36	18.0	8
5	5.62	2.80	36	24.0	5
6	6.81	4.25	36		5
		Heavy Weight			
1½	2.36	1.4	18, 24, 36	8.25	5
2	2.90	1.9	18, 24, 36	9.5	5
2½	3.18	2.4	24, 36	10.5	8
3	4.03	3.0	36	13.0	8
3½	4.57	3.6	36	15.0	8
4	5.12	4.3	36	16.0	8
4½	5.69	5.1	36	18.0	5

* Data from the Fibre Conduit Company.

Tile duct is made in single sections 18 in. long and in multiple sections 36 in. long. It is seldom used in the power station, but often in underground duct lines. Split tile is sometimes advantageous in cable rooms and manholes, as it can be put around the cables after they are in place. Multiple-tile duct is not suitable for installing heavy-capacity circuits as the fragile partitions are destroyed easily by a cable burn-out. For heavy-capacity duct lines, each duct, whether of tile or fiber, should be surrounded by at least 3 in. of concrete.

TABLE 11

Weights and Dimensions of Transite Conduit *

Nominal Inside Diameter, in.	Approximate Outside Diameter, in.	Net Weight, lb per foot	Radius of 45° and 90° Bend, in.	Standard Length, ft
		Standard Weight		
2	2.70	2.1	18, 24, 36	5
$2\frac{1}{2}$	3.20	2.6	18, 24, 36	5
3	3.74	3.3	24, 36	10
$3\frac{1}{2}$	4.24	3.7	36	10
4	4.74	4.1	36	10
$4\frac{1}{2}$	5.30	5.1	36	10
5	5.80	5.7	48	10
6	6.80	6.7	48	10
		Light Weight		
2	2.50	1.2	18, 24, 36	5
$2\frac{1}{2}$	3.00	1.5	18, 24, 36	5
3	3.54	1.8	24, 36	10
$3\frac{1}{2}$	4.04	2.0	36	10
4	4.54	2.3	36	10
$4\frac{1}{2}$	5.04	2.8	36	10
5	5.60	3.2	48	10
6	6.60	3.7	48	10

* Data from Johns-Manville International Corporation.

Installation of conduits in a building or dam must keep pace with the construction, and it is therefore necessary to prepare well-detailed conduit plans in advance of field work. Suitable conduit sizes for various groupings of wires and cables are given in Table 12.

20. Lightning Protection. The relative importance of lightning protection in various parts of the country is indicated by the number of thunderstorm days per year, as shown in Fig. 6. It generally is considered good insurance to provide lightning protection of major station equipment even in light-thunderstorm areas, as the cost of good protection is a small percentage of the cost of the equipment to be protected. Transformers, regulators, and breakers are among the important items of outdoor equipment which require protection.

Protection against direct lightning strokes is effectively provided by installing a shielding network of ground wires over the station. Each overhead ground wire affords protection over a considerable area below it. It usually is considered that all equipment is adequately protected from direct stroke if it is located within 45 degrees of a perpendicular dropped from the overhead ground wire. The overhead shielding network must be grounded very thor-

TABLE 12

Suitable Conduit Sizes for Various Groupings of Wires and Cables

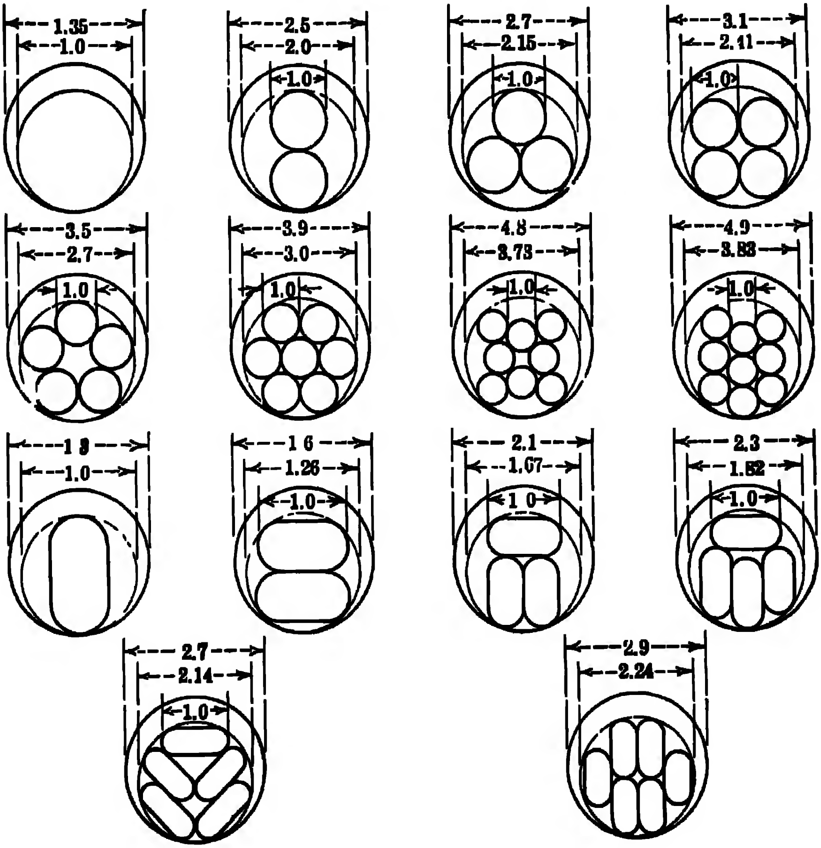
Diagram shows smallest equivalent diameter of group of wires and diameter of conduit in terms of diameter of a single wire.

Diameter of conduit is for runs from 50 ft with two 90° bends to 150 ft with one 90° bend.

For more difficult runs and wherever cables are larger than 1 in. in diameter, increase diameter of conduit by 15%.

Example: Size of conduit for four No. 1/0 wires. Diameter of No. 1/0, rubber-insulated, jacketed, 600-volt wire, from Table 5, = 0.72 in.

$$0.72 \times 3.1 = 2.232 \text{ in} \qquad \text{Use } 2\tfrac{1}{2}\text{-in. conduit.}$$



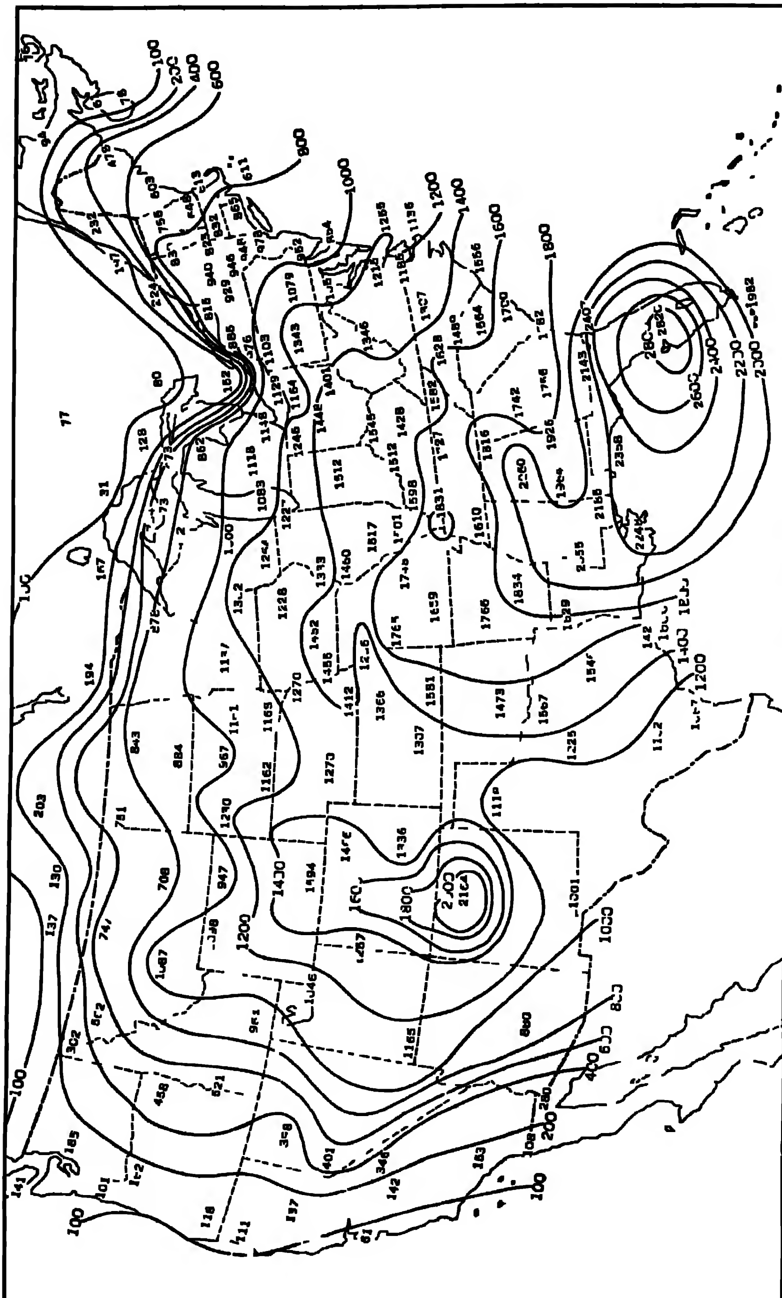


FIG. 6. Isoceraunics—total thunderstorm days in 30 years, 1904–1933. (From *Monthly Weather Review*, May 1935.)

oughly at many points. It would be ideal to suspend the overhead shielding network above the switchyard on steel structures entirely independent of the switching structures. In most cases, however, the cost of such an installation would be excessive, and the overhead ground network is supported by steel peaks atop the switching structures.

If ground wires are provided above the transmission lines, they should be connected to the network over the station. If not, it is good practice to extend the station shielding out over the transmission lines for at least ½ mile so that any lightning strokes to the transmission conductors where they are unshielded will have at least ½ mile in which to attenuate before entering the station.

Protection against surges entering the station as a result of lightning strokes to unshielded transmission lines or to ground wires or to adjacent objects is provided by the installation of lightning arresters at or close to the equipment to be protected.

For further information on lightning protection, see Section 28, Ref. 2 and Chapter 40, Section 17, Ref. 20.

21. Lightning Arresters. Many forms of arresters have been used. An early form consisted of small gaps between cylinders of nonarcing metal in series with a carbon resistor. Another was the aluminum cell or electrolytic arrester. It consisted of a protective gap and a series of aluminum trays or cups nested and separated by electrolyte, thus forming a series of electrolytic cells. When kept charged by closing the gap each day against normal 60-cycle voltage, an insulating film was maintained on the surfaces of the aluminum cups which was broken down only by an abnormal voltage surge. The insulating film re-formed immediately with the passing of the surge.

A modern lightning arrester consists essentially of a gap and a resistor of special characteristic connected in series between each line and ground. The gap breaks down and passes current when the voltage reaches the critical voltage at which the arrester is designed to discharge, and at lower voltages prevents any appreciable flow of current. The gap is designed to have a shorter breakdown time on impulses than the apparatus it protects, so as to discharge a surge before it has time to damage the apparatus. The resistor has a resistance characteristic roughly inversely proportional to the applied voltage so that it discharges rapidly during a voltage surge and promptly seals off the current flow after the surge has passed. The resistor has sufficient thermal capacity to absorb some energy from the incoming surge and thus reduce reflected voltages.

In applying an arrester to a power system the breakdown voltage of the gap should be as low as possible to secure best protection of equipment, but the 60-cycle reseal voltage must be higher than the maximum 60-cycle voltage to avoid destruction of the arrester. Generally a single-phase arrester connected from line to ground on each phase of a 3-phase system will give satisfactory protection and will withstand the usual 60-cycle overvoltages if the reseal voltage of the arresters is equal to about 105% of the system line-to-

line rated voltage for an isolated neutral system, and about 84% for a grounded-neutral system. Example: For a 115-kv system, a 121-kv (60-cycle reseal) arrester generally is applied if the system neutral is isolated, and a 97-kv (60-cycle reseal) arrester if the system neutral is grounded.

Actual 60-cycle overvoltages on both isolated-neutral and grounded-neutral systems vary considerably with system constants and methods of grounding. For important installations, therefore, the expected overvoltages should be calculated for the specific arrester location before specifying the arrester rating [8]. In cases in which important equipment is to be protected but in which unusually high 60-cycle overvoltages may be expected only on rare occasions, it is considered sound economy to apply an arrester that gives good protection against impulses, although at some risk of occasionally losing the arrester by abnormally high 60-cycle voltage.

Industry standard arrester characteristics are given in Section 28, Refs 4 and 5.

Arresters for rotating machines are of special design. The arresters should be shunted by special station-type capacitors to slope the incoming voltage surge unless the generator cables contain as much capacitance as 0.25 microfarad per phase.

Arresters should be located as close as practicable to the equipment to be protected. In a transforming and switching station the arresters usually are connected to the circuit close to the transformer terminals, since the transformer is usually the most expensive apparatus in the station and is most vulnerable. Often the arresters are mounted on the transformer case. The ground terminal of each arrester should be connected to the transformer case and thoroughly grounded

22. Grounding. The station grounding system performs three classes of service: (1) safeguarding of employees, (2) protection of circuits and apparatus, and (3) operation of relays. In all cases it is very important that a reliable, low-resistance, large-ampere-capacity ground connection be installed. Much assistance in the proper location of grounds can be obtained from United States Geologic and other soil maps. Large buried bodies of steel or iron, such as penstocks, abandoned steel sheeting, and water mains, often form excellent grounds, but in some cases apparently well-grounded objects and structures are found by test to have a considerable resistance to ground. For instance, a copper plate thrown into the bed of a mountain stream is often found to be surprisingly insulated from earth on account of the purity of the water and the rock formation below. It is, therefore, always necessary to select carefully the location of the grounds and to test them thoroughly, not only when they are installed but also periodically during their service.

A copper-plate ground is made by burying a plate of copper about 3 or 4 ft square sufficiently deep in the earth to lie in permanent moisture. Crushed coke or charcoal should be packed around the plate to a depth of about 2 ft above and 2 ft below, and the excavation should be filled with soil mixed with common salt. The ground wire should consist of stranded copper and should be well spread out over the plate and riveted and soldered.

The iron-pipe ground is very easily installed and very effective. Galvanized-steel, 1- to 3-in. pipes, without joints, should be driven to permanent moisture. Suitable points and driving caps are available. The ground wire, which should consist of copper stranded cable, should be connected to the pipe by suitable clamps, or may be soldered by plugging the pipe about 12 in. below the top, inserting a loop of the cable, and filling with a solder consisting of 75% lead and 25% antimony, which will not contract as it cools. Pipe grounds should not be placed closer to each other than 6 ft. At least two pipes should be used for every installation, and many more are preferable. In some instances, it is advisable practically to surround the station with ground pipes. No rule can be given as to maximum resistance allowable, but each pipe under average conditions would be expected to have a resistance of 10 to 25 ohms.

Ground mats of stranded copper cables are very common for important stations. Number 4/0 or 500,000-cir-mil bare cables are placed in trenches to form a grid with meshes about 10 ft square with all joints brazed. Where especially high-resistance soil is encountered it is sometimes necessary to haul in low-resistance soil for backfill or to surround the conductors with coke and soil. The mats should be located if possible where they will be permanently moist.

Provision for testing the various mats from time to time should be made by using insulated cables for the terminal connections and carrying them through test boxes or handholes so that each mat may be disconnected and tested separately. Ground resistances of a number of driven pipes or buried mats may be measured directly by the use of a J. G. Biddle "Megger" or an ohmmeter. The procedures are described in the instructions accompanying these instruments. The 3-point method by use of an ammeter, a voltmeter, and an a-c source is described in *Bureau of Standards Technological Paper* 108, "Ground Connections for Electrical Systems." The procedure is essentially as follows:

1. Obtain a source of a-c supply at about 110 or 220 volts single-phase and reduce it by means of a two-winding transformer to such voltage as will give about 10 to 50 amperes through two of the ground pipes in series. A two-winding transformer is essential to secure a testing voltage that is positively ungrounded.
2. Select a voltmeter and an ammeter of proper scales.
3. Letter the ground pipes A, B, C, D, etc.
4. With one side of the test circuit connected to A and the other through the ammeter to B, read amperes through A and B and voltage across A-B. The volts divided by the amperes gives the resistance in ohms, R_{AB}, of pipes A and B.
5. Repeat for pipes B and C obtaining R_{BC}, and for pipes C and A obtaining R_{CA}.
6. Compute resistances of A, B, and C from equations:

$$\text{Resistance of } A = R_A = \frac{R_{AB} - R_{BC} + R_{CA}}{2}$$

$$\text{Resistance of } B = R_B = \frac{R_{BC} - R_{CA} + R_{AB}}{2}$$

$$\text{Resistance of } C = R_C = \frac{R_{CA} - R_{AB} + R_{BC}}{2}$$

7. Repeat entire process with pipes B, C, and D, securing a check on R_B and R_C and new value R_D. By proceeding through all the pipes in this manner the last test should include A, giving a check on R_A.

Present practice is to connect all ground mats to the powerhouse and switching station grounding buses through the test boxes mentioned in the previous paragraph. The overhead grounding systems of the transmission lines and the switching station and usually the bases of the nearest steel towers also are connected to the grounding buses. Lightning arrester grounds are now connected to the station ground bus, but it is good practice also to provide very low-resistance ground mats at the arresters to avoid excessive voltage rise on the ground bus during discharge. A grounded protective mat 4 to 6 in. below the walking surface of the switching station improves the protection of employees by reducing the potential gradient during lightning-arrester discharge and during heavy current flow from the overhead grounding system to ground.

Items to be grounded for the safeguarding of employees include: frames and cases of all generators, transformers, breakers, disconnects, motors, controllers, switchboards, instruments, relays, cabinets, cubicles, cable shielding and sheaths, conduits, and all noncurrent-carrying metal parts of all apparatus. The grounding of generator and transformer neutrals for protection of the circuits and apparatus and for the operation of relays is discussed in Chapter 40, Section 17, Ref. 20.

For further information on grounding of power systems, see Sec. 28 of this chapter, Refs. 6 and 8.

23. Auxiliary Power Supply. Although the maintenance of an uninterrupted supply of auxiliary power is not so important for a modern hydroelectric plant as it is for a steam plant, it nevertheless is considered good practice to provide several alternative sources. Such sources as the main generator leads, the generator bus, the low-voltage terminals of the step-up transformer, the transmission-voltage bus, an independent distribution circuit passing near the station, a house generator on a main generator shaft, and a house generator driven by a small water wheel are among those commonly used. A source such as the generator bus or the generator-transformer circuit where a generator bus is not used receives its supply from either the generator or the power system and is therefore more firm than a single source. It is good practice to provide two duplicate, full-capacity transformers so that reasonable continuity can be expected even while switching main generators, transformers, and lines. An outside source independent of the station itself is convenient to maintain lighting, pumping, and gate operation in case the entire plant should be shut down for maintenance or during filling season, or during light-load periods.

An emergency source often is needed to operate spillway gates, valves, and sump pumps, and to provide lighting in case of a major outage that might isolate the station from all normal electrical connections. Gasoline-electric generators frequently are used as stand-by sources for this purpose. The

station storage battery also is used as an emergency supply for lighting, for essential lubricating pumps, and for certain important valve operations such as the closing of penstock valves.

Paralleling of the various sources usually is avoided because of the heavy short-circuit stresses that would be involved and in some cases because of the circulating currents that would result. It is common practice to accept a momentary interruption when switching from one source to another rather than accept the current surges that might result from momentary paralleling.

The amount of auxiliary power usually required for motor-driven auxiliaries for lighting and for heating amounts to 0.5 to 3% of the station capacity. The power demand varies from 20 to 30% of the connected load. It is common practice to use 2200-volt, 3-phase service for motors larger than 100 hp; 440-volt, 3-phase for motors from 100 to 1 hp and for large heaters; 220-volt single-phase for fractional-horsepower motors and small heaters; and 230-115-volt, single-phase for lighting. These, however, are not fixed rules. Often, for example, a few motors considerably larger than 100 hp, if not too far from the source of supply, are operated at 440 volts. Feeder-voltage regulators often are needed where the auxiliary power supply is taken from the main power circuits.

Items that should be considered in estimating the auxiliary power load include: lock gates, valves, pumps, heaters; spillway gates, sluice gates, gantry cranes, trash racks, ice prevention; powerhouse cranes, draft-tube gate cranes, unloading cranes, derricks; exciters; governor oil pumps, lubricating oil pumps, sump pumps, dewatering pumps, fire pumps, house service pumps; air compressors; lubricating and insulating oil purification and pumping; generator and transformer cooling fans; battery charging; machine shop tools; heating, ventilating, and air conditioning.

24. Auxiliary Power Distribution. A single-line wiring diagram of a typical auxiliary power distribution system is shown in Fig. 7, simplified for presentation. Two duplicate transformers constitute the normal supply. Each is connected to the low-voltage leads of a main step-up transformer bank and hence each has two sources: one from the power system, the other from a generator. A third, smaller source consists of a transformer stepping down from a distribution line in the vicinity. A fourth, small source consists of a gasoline-driven generator in the station. Connections of this generator are such that it can be loaded by the air-conditioning system for periodic testing without synchronizing it to the auxiliary power system. The supply sources and the main feeders to centers of distribution all are controlled by a main auxiliary power switchboard. Distribution boards consist of a unit board for the auxiliaries of each generating unit, a common board for cranes, sump pumps, air compressors and other auxiliaries not associated with any individual generating unit, and various smaller distribution cabinets and boards.

Parallel operation of the two main supplies is prevented by interlocks and by double-throw switches at destinations of main feeders. Every bus and every bus section is provided with two sources of supply.

Inspection and maintenance of all parts of the power distribution system may be accomplished without seriously curtailing generation The main bus

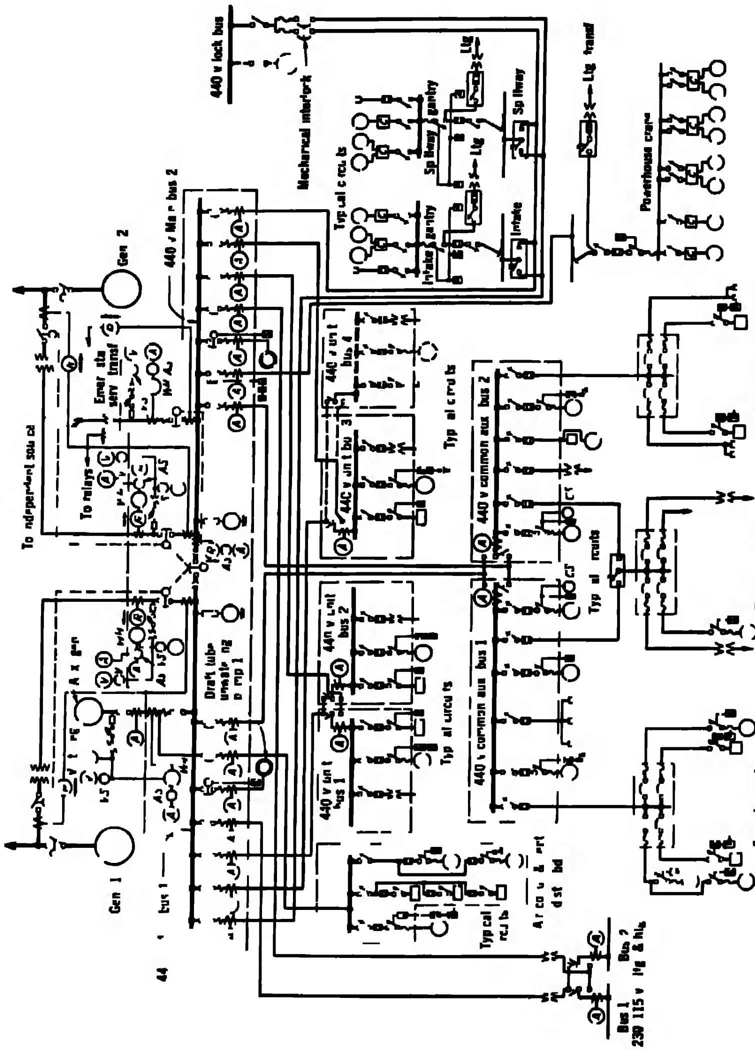


FIG 7 Typical auxiliary power diagram

is in two essentially duplicate sections, either of which with its supply and feeder breakers can be retired, leaving the other section and all essential auxiliaries in operation The bus-sectionalizing breaker is of withdrawal type and can be withdrawn for inspection, leaving both bus sections in operation Each unit bus may be retired whenever its generator is shut down Either

section of the common bus may be retired without shutting down essential auxiliaries. Each circuit from the various distribution centers may be isolated individually by its own disconnecting switch.

25. Motors and Controllers. The 3-phase squirrel-cage induction motor, simplest of all motors, is applicable to a great majority of auxiliary drives. It has no moving or sliding connections such as slip rings, commutators, and brushes, and it does not require any built-in starting devices such as single-phase motors require. By variations in rotor design, the starting torque at rated voltage can be varied from 50% to 300% of full-load rated torque, and the starting current at rated voltage can be varied from 400% to 700% of full-load rated current. Special torque and current characteristics, however, are obtained usually at the expense of low efficiency and low power factor. The general-purpose motor, designed for about 170% starting torque, 650% starting current, and 260% pull-out torque, has good efficiency and power factor and is well adapted to most of the station auxiliaries such as pumps, fans, crane travel, gate hoists. The rather high starting torque is not required by pumps or fans but can be accepted if the machines are designed for it. The high starting current is not objectionable since most of the auxiliaries are comparatively close to the power source. These auxiliaries usually are started at full voltage, so that the control equipment is simple and rugged. It usually consists of a fused knife switch for isolating the entire circuit and for short-circuit protection, and a magnetic contactor with thermal overload trips for starting and stopping and for running protection.

The wound-rotor or slip-ring induction motor is next to the squirrel-cage type in simplicity and ruggedness. Bringing the rotor winding out through slip rings and brushes permits the inserting of variable amounts of resistance in the rotor circuit while the motor is in operation. This not only permits changing the torque characteristics over a wide range but also provides speed control and dissipates most of the accelerating and speed-control losses outside the motor. Such motors are adaptable to accessories such as hoists, derricks, winches, and transfer cars which require heavy acceleration, frequent starting, or variable speed. The control includes in addition to the primary control required for the squirrel-cage motor a secondary control which consists of a resistor and a drum controller or set of magnetic contactors for inserting various amounts of resistance in the rotor circuit. The resistance steps can be designed to give uniform steps of speed either for a varying-torque load, such as a pump or fan or for a constant-torque load such as a hoist.

Synchronous motors are justified in some cases to improve the power factor of the auxiliary power system, but in a hydroelectric station this is seldom necessary. With induction motors properly applied so that they operate at or near their rated loads, power-factor correction usually is not required. When used, synchronous motors are applied principally to air compressors since they start easily when unloaded and since they operate at constant speed. The higher cost, the more complicated control, and the need of exciters make

synchronous motors less desirable than induction motors, although in some ratings they are more efficient.

Direct-current motors of the series, shunt, or compound type are used for remote synchronizing, for emergency bearing lubricating pumps, penstock valves, and other small but important auxiliary services driven from the control battery. Fractional-horsepower motors usually are started at full voltage, and integral-horsepower motors are started generally by resistance starters.

26. Lighting. The lighting load can be expected to include many of the following items, depending upon the general type of development: dam inspection tunnels, roadways, spillway gates, sluice gates, intake gates; lock chamber, decks, control house, navigation lights; powerhouse generating room, control room, electrical bay, service bay, turbine pits, public areas; switchyard walking area, overhead equipment; yard and grounds, roadways, parking areas; floodlighting of dam, spillway, powerhouse, and switchyard.

It is good practice to provide two sources of power, such as two transformers from the auxiliary power bus, although it is not necessary that each be full-capacity since the lighting load usually can be curtailed temporarily in case of failure of a source. A voltage regulator usually is provided unless the source has voltage regulation. The commonly used 230-115-volt, 3-wire, single-phase distribution system is usually adequate, although it is usual to run 2300-volt or 440-volt, 3-phase circuits to large, distant load centers such as the switchyard and the dam. From the lighting switchboard 3-wire circuits are carried to the various distribution cabinets located near load centers. Switching is provided locally. Large areas such as the generator room, the switchyard, roadways, and floodlighted areas are controlled by contactors in the distribution cabinets. Photoelectric switches often are used to control such outdoor lighting as switchyards, dam roadways, and floodlighting. Low voltage, such as 32 volts, often is used for convenience outlets in turbine pits and near entrances to scroll cases and draft tubes where very damp conditions are encountered.

Emergency lighting arranged to operate on the battery or other dependable source in case of failure of the normal source should be provided for vital parts of the station such as control room, governors, auxiliary power switchboards, passages, and stairways. Where the control battery serves as emergency source it usually is given sufficient capacity to supply the emergency lighting for at least an hour. Several wiring and switching schemes have been used, four common ones being illustrated in Fig. 8. Schemes *A* and *B* have the advantage that small lamps may generally be used for the emergency lighting, thus reducing the duty on the battery and charger. Schemes *A* and *C* have the advantage that no automatic throw-over is needed. Schemes *B* and *D* have the advantage that the battery charger can be smaller since the emergency lights normally burn on the normal source. Schemes *C* and *D* have the advantage that burn-outs of emergency lamps are detected more easily because the emergency lights are a part of the normal lighting system.

Conditions often require combinations of these schemes. For instance, Scheme *D* may be used generally throughout the station and Scheme *C* may be used for a very few lights at the main control board where they are so vital that it is felt that they should not depend upon operation of the automatic throw-over switch.

When a room contains both normal lights and emergency lights, it is wise to arrange the local switching so that the normal and emergency lights must be switched on or off at the same time. When the normal lighting voltage is

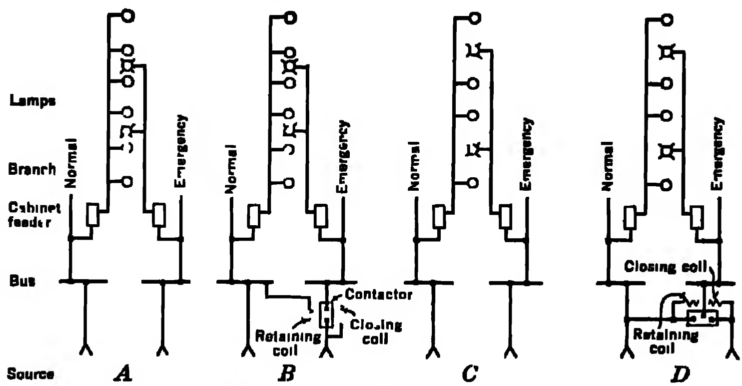


FIG 8 Emergency lighting schemes (*A*) Emergency lights in addition to normal lighting, always connected to emergency source (*B*) Emergency lights in addition to normal lighting, automatically connected to emergency source when normal source fails (*C*) Emergency lights a part of normal lighting, always connected to emergency source (*D*) Emergency lights a part of normal lighting, automatically connected to emergency source when normal source fails.

115 volts and the emergency voltage is 250 volts, obviously two kinds of lamps must be used or the emergency lamps must be wired in groups of two in series. The former generally is more practicable, and in this case the sockets should be marked distinctly as a precaution against inserting a 115-volt lamp in a 250-volt socket.

Estimates of the wattage required for the illumination of various areas of a station can be obtained from the relation:

$$W = \frac{IA}{ELD}$$

where W = aggregate wattage of lamps, watts;
I = illumination intensity, foot-candles;
A = area of floor, square feet;
E = efficiency of utilization, depending on type of unit, shape of room, and character of interior finish, percentage;
L = lamp initial efficiency, lumens per watt;
D = depreciation factor.

In making very rough calculations for ordinary conditions, the constants of Table 13 and the lamp data of Table 14 may be used.

TABLE 13

ILLUMINATION DESIGN DATA

Room	Illumination Intensity, foot-candles	Type of Fixture	Maintenance Factor	Efficiency of Utilization, per cent
Aisles, corridors, stairways	5	Glass	0.70	0.25
Auxiliary switchboard rooms	10	Glass	.70	.30
Battery rooms	10	V.P. glass	.70	.30
Bus rooms	5	Glass	.70	.30
Control room	25	Indirect	65	.20
Control switchboards *	20	Indirect	.65	.20
Elevators	10	Cove	.60	.35
Equipment rooms	10	Glass	.70	.30
First-aid room	40	Indirect	.65	.35
Gas engine room	10	V.P. steel	.70	.40
Generator room †	15	Highbay	.70	.50
Intake and spillway deck	1	Handrail	.60	.20
Laboratory	40	Glass	.70	.30
Locker room	10	Glass	70	.20
Machine shop	30	Glass-steel	.70	.40
Motor-generator room	10	Glass	70	40
Offices, average	20	Indirect	.65	25
close work	40	Indirect	.65	25
drafting	50	Indirect	.65	25
Oil purification room	20	V.P. steel	.70	35
Oil storage	5	V.P. steel	.70	25
Reception room ‡	10	Indirect	.65	20
Storage rooms	10	Steel	.70	40
Storage yard	1	Steel	.65	20
Switchyard, walking area	2	Glass	.65	.30
switching structure	5	Glass	.60	.35
Toilet rooms	10	Glass	.70	.35
Transformer rooms	10	Glass	.70	.30

* Measured perpendicularly to the panels.
† Intensified by lights on crane during maintenance.
‡ Displays intensified locally.

27. Communication Systems. Every hydro plant should have telephone communication facilities with other plants on the same power system and with the system load dispatcher. All but the smaller plants also require an intercommunication system between various parts of the plant and between the plant and the headworks.

For communications between plants and with the load dispatcher, four channels are common: wire line leased from the local telephone company, wire

TABLE 14

STANDARD LAMP DATA

Watts	Bulb	Base	Volts	Description	Rated Hours	Initial Lumens per Watt
25	A-19	M	115	F	1000	10.4
25	A-19	M	230	F	1000	8.0
40	A-19	M	115	F	1000	11.6
50	A-19	M	115	F	1000	13.2
50	A-19	M	230	F	1000	9.5
60	A-19	M	115	F, B	1000	13.9
75	A-21	M	115	F, C	750	14.9
100	A-21	M	115	F, C	750	16.2
100	A-23	M	115	B	750	16.2
100	A-23	M	230	F	1000	12.4
150	PS-25	M	115	F, C, B	750	17.3
200	PS-30	M	115	F, C	750	18.2
200	PS-30	M	115	B	1000	18.2
200	PS-30	M	230	F, C	1000	14.7
300	PS-35	M	115	F, C	750	19.5
300	PS-35	S, G	115	F, C, B	1000	18.8
300	PS-35	S	230	C	1000	16.2
300	PS-35	G	230	F, C	1000	16.2
500	PS-40	G	115	F, C, B	1000	19.7
500	PS-40	G	230	F, C	1000	17.6
750	PS-52	G	115	F, C, B	1000	19.9
750	PS-52	G	230	F, C	1000	18.1
1000	PS-52	G	115	F, C, B	1000	21.0
1000	PS-52	G	230	F, C	1000	19.1
1500	PS-52	G	115	F, C	1000	22.3
1500	PS-52	G	230	C	1000	19.0

A-19 = shape A, diameter
F = inside frosted
B = silver bowl
C = clear
PS-25 = shape PS, diameter $^{25}/_{8}$ in.

line owned by the power company, carrier current on the transmission conductors (power-line carrier), and carrier current on the telephone wire line (wire-line carrier). Many large systems employ several of these channels to meet various situations. A telephone turret or compact switchboard usually is provided on the control-room operator's desk for manually switching all outside calls.

For intercommunications within and near the plant, a small magneto or push-button system is often adequate for a small plant. A large plant usually requires either a manual or an automatic battery-operated exchange. The automatic usually is preferred so that most of the calls can be dialed directly from phone to phone without requiring the attention of the control-room

operator or necessitating a separate operator. The interphone system may be equipped with such special services as dispatching circuits, lines to the local telephone company's exchange, code call, executive right-of-way, and conference.

The telephone equipment room should be planned carefully. For automatic equipment, the delicate contacts should be protected from dust by providing linoleum or other dustproof floor covering, painted walls, and air filters in the ventilating system. Paint for the telephone room should not contain turpentine, because turpentine fumes are known to cause a nonconducting glaze on relay contacts.

Hand sets about the station have the advantage that vibration in building walls are not transmitted. Jacks and portable phones are used in such locations as tunnels, hoist wells, and scroll case entrances, where only periodic use is indicated. Weatherproof phones with moistureproof wiring and weathertight cases are necessary for damp locations. For noisy locations, noiseproof transmitters or soundproof booths are available.

Radio also is used by power systems for automatically transmitting rainfall and stream-flow reports from remote points on the watershed to the central office. Manually operated radio equipment also is used to some extent for reporting and suppressing forest fires and for communication with line crews.

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CHAPTER 43

TRANSMISSION LINES

By Raymond A. Hopkins and Harry O. Weber†*

1. Transmission Lines. The hydroelectric generating station is usually located at a considerable distance from the distribution system into which it feeds; hence the connecting transmission lines are of great economic importance. Unlike the usual steam plant which radiates its energy over a number of comparatively small feeders, the hydroelectric plant usually delivers its entire output over two or three heavy transmission lines. The cost and performance of the lines therefore have an important bearing on the cost of energy delivered at the load center.

2. Right-of-way. The proper location of the right-of-way over which a transmission line is to be constructed is an economic problem of great importance. Careful reconnaissance work is needed to determine the best location. In flat and rolling country, aerial observation and photographic mapping are now used to advantage. In rough country, an aerial survey is of somewhat less value, although it may aid in the selection of the most direct route and in securing the necessary information for the purchasing of right-of-way. Accessibility for delivery of materials and for patrolling is a controlling factor. The detailed survey following the reconnaissance should locate lakes, swamps, hills, towns, railroads, power and communication lines, and legal property lines. Profiles should be run for determining tower locations and heights to give the necessary clearance of the cables above the ground. The maps and profiles should show probable tower locations as selected by the field party.

The right-of-way for an important transmission line is sometimes owned by the power company, although in some cases the power company may preferably accept easements over certain parcels of privately owned property. The width of the right-of-way is determined by the ultimate number of circuits that may be run. It is desirable to place parallel tower lines so far apart that a falling tower of one line will not strike the other line. The edge of the right-of-way should be at a safe clearance distance, roughly 2 in. per 1000 volts, beyond the extreme side-swing position of the center of the span. The right-of-way should be cleared of trees, and it may be advisable to cut trees on adjacent property to prevent the possibility of their falling on the lines.

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3. Frequency and Phase. The frequency of the transmission line is generally determined by the frequency of the power system with which the line will be connected, as this consideration usually outweighs the various slight advantages of one frequency over another (see Chapter 40, Section 7).

Three-phase transmission is practically standard for all important lines. Distribution lines are sometimes single-phase, 2-phase, or direct-current, as governed by local conditions; but the tendency, for such lines, also, is toward the uniform use of the 3-phase system. The principal advantages of the 3-phase system are high copper economy and small number of conductors. (See Chapter 40, Section 8.)

4. Number and Arrangement of Circuits. For maximum reliability it is desirable to divide the transmission system into at least two parallel circuits, each of such capacity that one may be retired for inspection and maintenance without curtailing the delivery of energy. Where a large block of power is transmitted a long distance, it becomes quite necessary to use more than one circuit in order to avoid excessive reactance drop.

A separate pole line or tower line is preferable for each circuit, in order to provide greater safety for the maintenance men and also to prevent trouble on one circuit from affecting the other. The latter consideration may, in extreme cases, justify routes sufficiently separated to prevent having both lines in the same storm area. Where more than two circuits are used, double-circuit towers are often considered satisfactory and are less expensive than twice the number of single-circuit towers.

Clearances over highways, railroads, and other wires are specified in the rules of the National Safety Code. The lower the towers the more reliable will be the line from the standpoint both of mechanical strength and of freedom from lightning strokes.

The configuration of the wires is generally triangular, flat-vertical, or flat-horizontal. The triangular configuration is adapted to the lower and moderately high voltages, particularly where the conductors are supported by pin-type insulators. It may be symmetrical, that is, with equal spacings between conductors, or unsymmetrical. The flat-vertical configuration is adapted to two-circuit towers for high voltages. The middle conductor is usually offset from the plane of the other two by a distance of about one third the spacing, to reduce the possibility of trouble from sleet and snow. The flat-horizontal configuration is adapted to single-circuit towers for high voltages. On very long lines it is sometimes advantageous to transpose at least twice, for the purpose of balancing the reactance and capacity equally among the three conductors and between the conductors and ground.

Spacing between conductors cannot be determined by any fixed rule. It should be roughly proportional to the voltage but also depends upon the configuration, type of insulator, length of span, conditions of loading, and local regulations. A larger spacing is usually required horizontally than vertically. Larger horizontal spacing is required for suspension insulators than for pin insulators.

Ground conductors are often used on steel tower lines as a means of protecting the power conductors from lightning. The ground conductor should be of high-strength steel and should be either galvanized or copper-covered. The connections at the towers should be firm and so arranged as to avoid wear from the swinging of the conductor [6].

5. Insulators. The conductors must be attached to the supporting structures by means of insulators designed to withstand safely the mechanical stress and electric potential. Insulators are usually made of vitrified, glazed porcelain. Glass has also been used to a limited extent for high-voltage lines and quite generally for low-voltage distribution lines and for telephone lines. The two general types of insulators are pin-type and suspension-type.

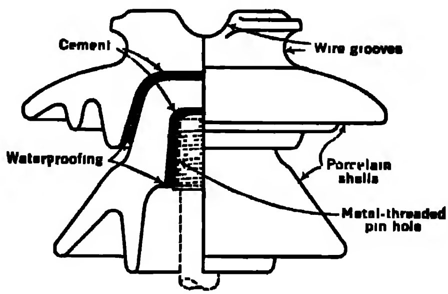


Fig. 1. Typical pin insulator. (Locke Insulator Corp.)

The pin insulator is intended for mounting on a wood or steel pin and is provided with a groove at the top for attaching the conductor. As it is not practical, from the manufacturing standpoint, to make porcelain of greater thickness than about ¾ in., and as this thickness is not suitable for a voltage of more than 10,000 to 15,000 volts, the high-voltage insulator is made by assembling several concentric shells as shown in Fig. 1. The shells are cemented together with Portland cement, the surfaces in contact with the cement being sanded before firing, to give a strong bond. Objectives sought in the design of the pin insulator include maximum leakage distance with minimum leakage area, greater flashover voltage than puncture voltage, avoidance of corona formation, maximum mechanical strength, and endurance of extreme and sudden temperature changes and weather conditions. Although the pin insulator is made for voltages as high as 88,000 volts, it is usually more economical to use the suspension type for voltages above 60,000 volts.

Insulator pins for voltages of 15,000 volts and less are commonly made of wood, such as locust, oak, birch, maple, hickory, and eucalyptus. Wood pins are usually treated with paraffin, oil, or creosote. For higher voltages, a steel pin is generally required on account of the mechanical strength necessary. To secure a tight fit between the steel pin and the porcelain insulator without excessive pressure at spots, various devices have been used, as threading and splitting the pin top, providing a wire spring thread, providing a threaded

wood or lead top, cementing into the insulator, or providing a separate metal thimble which is cemented into the insulator at the factory and threaded on to the pin in the field. The metal thimble now is almost universally used for voltages of 23 kv and over. It insures a tight fit and avoids corona discharge in the pinhole.

Suspension insulators are generally shaped like disks and are intended for connecting together in strings to suspend the conductor (see Figs. 2 and 3).

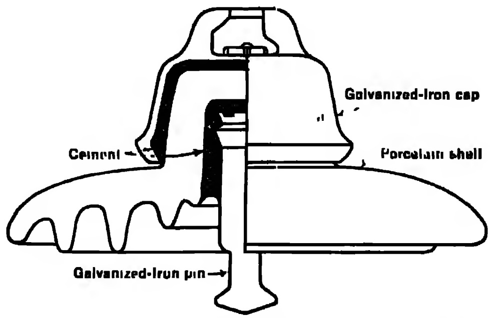


FIG. 2. Typical suspension insulator. (Locke Insulator Corp.)

A string may be used either vertically, as for an intermediate support, or nearly horizontally, as in a dead-end or strain support. Where great mechanical strength is required or where greater safety is desired a group may be formed by arranging several strings in parallel, care being taken to distribute the load equally among the various strings by means of equalizing bars at the ends. The distribution of the voltage stress among the various units of a

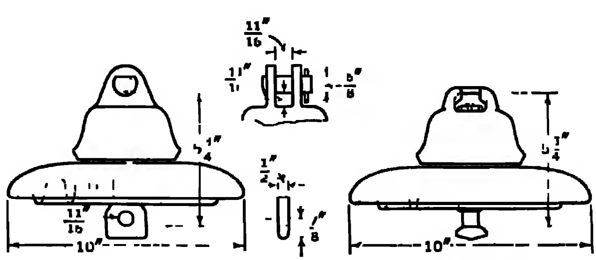


FIG. 3. Clevis-type and socket-type suspension insulators. (Locke Insulator Corp.)

string of suspension insulators is not at all uniform. The end units receive more potential than the intermediate units, and the conductor end receives more than the support end. This is caused by a distortion of the electrostatic field as influenced by the condenser action of the metal hardware between units. For high potentials this phenomenon is of serious importance, as the unit next to the conductor may receive a potential exceeding its rating. A

number of installations have been made with various forms of electrostatic shields at each end of the string for grading the voltage. An example of insulator grading and its effect are shown in Fig. 4.

Tests of insulators are of two classes: design tests made on a limited number of insulators to check the design, and routine tests made on all insulators to detect defects. The routine tests should include flashover of each shell

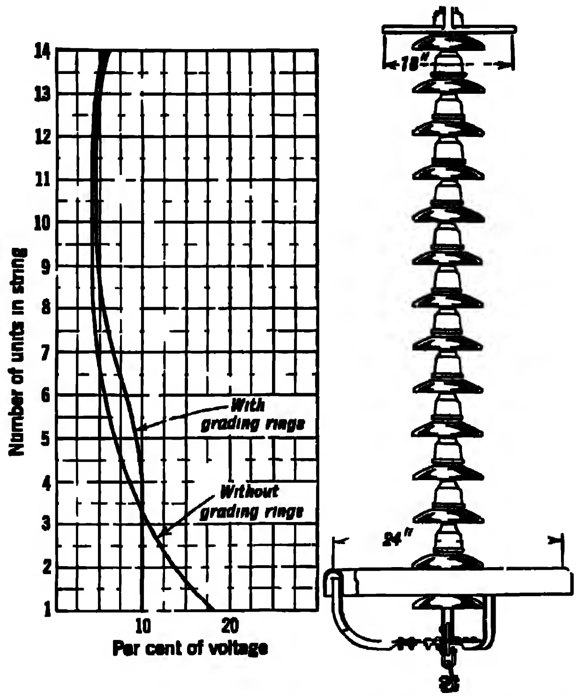


Fig. 4. Distribution of voltage on insulator string with and without grading rings. (Locke Insulator Corp.)

before assembly, flashover of the completed insulator, and mechanical strength of the completed insulator. It is not recommended that any insulator be tested to over 50% of its ultimate mechanical strength before being put into service. A higher test is likely to cause permanent injury and, furthermore, is not needed to detect most defects. A selected few insulators, however, should be tested to destruction. For important installations, it is advisable to apply a combined mechanical and electrical test to destruction, as both electrical failure and mechanical failure generally occur at lower values under these conditions. For standard conditions of test, reference should be made to the Standards of the American Institute of Electrical Engineers.

6. Corona. Corona is the breaking down of the air as a dielectric near the surface of a conductor when the potential gradient exceeds a certain criti-

cal value. The phenomenon is accompanied by a bluish light and a hissing sound. It is often manifest in electrical machinery and in station wiring unless precautions are taken to guard against it. On high-voltage transmission lines it often becomes a factor in limiting the diameter of conductor or some other feature of design, as the power loss increases very rapidly with increase in voltage above the critical value Very extensive experimentation by F. W. Peek, Jr., and others has resulted in the empirical formulas given below, which define both the critical voltage and the value of power loss.

The disruptive critical voltage is the voltage at which corona becomes apparent. During fog, rain, sleet, or snow, the critical voltage is from 10 to 20% lower than in fair weather. The critical voltage and power loss for the conductors of a transmission line under conditions of fair weather are:

$$E_0 = 107\frac{17.9b}{459 + t}\, md \log_{10}\left(\frac{2D}{d}\right) \qquad [1]$$

$$P = 0.039\frac{459 + t}{17.9b}\sqrt{\frac{d}{2D}}\,(f + 25)(E - E_0)^2 \qquad [2]$$

where E_0 = disruptive critical voltage, in effective kilovolts between conductors;
b = barometric pressure in inches of mercury at 32 degrees Fahrenheit corresponding to the altitude (30.00 at sea level, 28.86 at 1000 ft, 27.77 at 2000 ft, 26.75 at 3000 ft, 25.76 at 4000 ft, 24.78 at 5000 ft, 23.85 at 6000 ft, 22.98 at 7000 ft, 22.12 at 8000 ft, 21.28 at 9000 ft, 20.50 at 10,000 ft);
t = temperature of air, in degrees Fahrenheit;
m = irregularity factor of conductor surface (1.0 for polished wire, 0.98 to 0.93 for weathered wire, 0.87 to 0.83 for stranded cable);
d = diameter of conductor, in inches;
D = distance between centers of conductors, in inches (for irregular spacing $a, b, c,\quad D = \sqrt[3]{abc}$);
P = power loss per mile of line, in kilowatts;
f = frequency, in cycles per second;
E = operating voltage, in effective kilovolts between conductors.

7. Voltage and Conductor Size. The choice of voltage and conductor size, though usually modified by such local considerations as voltage regulation and connecting lines, nevertheless rests fundamentally upon pure economics as expressed by Kelvin's law. As the voltage increases, the power loss decreases, and the cost of terminal equipment increases. As the conductor size increases, the power loss decreases, but the cost of conductor increases. There is therefore an economical voltage and conductor size for which the total annual cost is the minimum.

Professors F. K. Kirsten and E. A. Loew, in *University of Washington Experimental Station Bulletin 32*, have presented a simple method of arriving simultaneously at the economical voltage and conductor size by making use

of the relation between voltage and conductor size as given in Mr. Peek's law of corona, on the assumption that a properly designed line will operate at just a safe margin below the corona voltage. Through the courtesy of the authors, a brief outline of this method is presented below, with some slight modifications consisting principally of expressing the conductor size in terms of area instead of diameter. The method is also extended to take account of various kinds of conductors, such as solid, stranded, steel-reinforced, rope-core, and hollow; various conductor spacings; and various substation layouts.

The notations and equations, with explanatory notes, are as follows:

Notation

P = root-mean-square value of power delivered by the line, in kilowatts. This must be determined from an exhaustive study of river flow and load-demand conditions. It is the rms ordinate of the average daily load curve.

p = average power factor at the load end, expressed as a decimal.

E = voltage between wires at load end, in kilovolts.

I = rms equivalent single-phase current in amperes $= \dfrac{P}{pE}$.

E_0 = disruptive critical corona voltage, in kilovolts between wires.

l = length of line, in miles.

a = cross-sectional area of conducting material of conductor, in circular mils.

S = spiraling factor of conductor, as follows:

Solid conductor	1.00
Stranded conductor	1.02

s = stranding factor or ratio of over-all diameter of conductor to diameter of equivalent solid conductor as follows:

Solid conductor	1.00
Standard stranded conductor	1.15 *
Steel-reinforced stranded conductor	1.23 *
Rope-core stranded conductor	1.35 *
Hollow stranded conductor	1.35 *

ρ = resistivity of conductor material, in ohms per mil-foot as follows:

	25° C	65° C
Hard-drawn copper, 97.3%	10.87	12.54
Hard-drawn aluminum, 61%	17.33	19.99

e = energy cost, in dollars per kilowatt-hour at receiving end of line.

c = conductor cost, in dollars per mil-foot of conducting material. The following figures are fair estimates:

* These values depend on the design of the cable but are sufficiently accurate for this purpose.

Solid copper	45.4×10^{-8}
Stranded copper	52.2×10^{-8}
Stranded aluminum	28.9×10^{-8}
Steel-reinforced aluminum	30.1×10^{-8}
Rope-core copper	55.4×10^{-8}
Hollow copper	59.0×10^{-8}

r = rate of interest and other fixed charges, expressed as a decimal

K = constant determined from the relation between line voltage and cost of terminal equipment, as expressed by the equation

$$\text{Cost} = K_1 + KE^2$$

The variable part of the cost, namely KE^2, is the only part to be considered. Careful estimates of typical substation layouts,

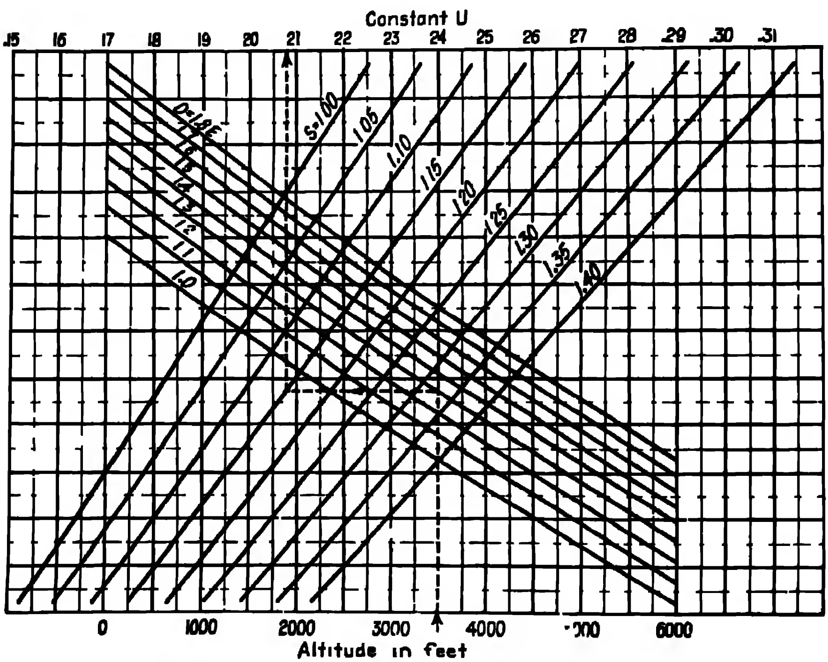


Fig 5. Relation between voltage and conductor size.

including transformers, breakers, disconnecting switches, arresters, and structures at both ends of the line, show that K varies from 8 for a simple layout to 13 for a complex layout.

$U = \dfrac{E}{\sqrt{a}}$ = constant obtained from curves of Fig. 5 and represents the

relation between voltage and conductor size to allow a safe margin below the corona voltage. The curves are obtained from the equation

$$U = \frac{E}{\sqrt{a}} = 0.00258sb \log_{10}\left(\frac{2000D}{s\sqrt{a}}\right)$$

which is derived from the corona equation, 1, by the following substitutions:

$$E = 0.85E_0$$
$$t = 77^\circ \text{ F.}$$
$$m = 0.85$$
$$d = 10^{-3}\, s\sqrt{a}$$

To use Fig. 5, select the average altitude of the line, move upward on the figure to intersection with curve representing desired relation between $\dot{D}$ and E, thence horizontally to intersection with curve s corresponding to type of conductor, thence upward to scale of U.

F, G, H = constants as defined by the three cost equations given below:

L_1 = annual cost, in dollars, of energy wasted in resistance of conductors.

L_2 = annual fixed charge, in dollars, on conductors, chargeable to their size.

L_3 = annual fixed charge, in dollars, on terminal equipment, chargeable to line voltage.

The Three Cost Equations

$$L_1 = \text{resistance} \times \text{current}^2 \times \text{hours} \times \text{energy cost}$$

$$= 5280\rho S\frac{l}{a} \times I^2 \times 8760 \times e10^{-3}$$

$$= (46253\rho Sle)\frac{I^2}{a}$$

$$= F\frac{I^2}{a}$$

$$L_2 = \text{quantity} \times \text{conductor cost} \times \text{rate}$$
$$= 5280la \times 3c \times r$$
$$= (15840lcr)a$$
$$= Ga$$

$$L_3 = \text{constant} \times \text{voltage}^2 \times \text{rate}$$
$$= K \times E^2 \times r$$
$$= (KE^2r)a$$
$$= Ha$$

Statement of Kelvin's Law

$$L_1 = L_2 + L_3$$

$$F\frac{I^2}{a} = Ga + Ha$$

$$a^2 = I^2 \times \frac{F}{G + H}$$

$$a^2 = \frac{E^2I^2}{U^2} \times \frac{F}{G + H}$$

$$a = \sqrt[3]{\frac{P^2}{p^2U^2} \times \frac{F}{G + H}} = \text{economical conductor size} \qquad [3]$$

$$E = U\sqrt{a} = \text{economical voltage} \qquad [4]$$

Mechanical considerations also affecting choice of conductor, such as weight and strength of conductor, number and cost of towers and foundations, and number and cost of insulators, are discussed in Section 15.

8. Fundamental and Derived Constants. The fundamental constants of a transmission line are its resistance, reactance, impedance, conductance, susceptance, and admittance. The constants are independent of voltage, current, and power factor, and are determined largely by the material, size, and spacing of the conductors. Many tables are available, giving the values of these constants, of which those contained in *Electrical Characteristics of Transmission Circuits*, by William Nesbit, are especially convenient [4]. The following paragraphs further define the constants and give equations from which they may be computed if desired.

Resistance, R, is expressed in ohms of one conductor and is determined from the equation:

$$R = 5.28\rho S\left(\frac{l}{a}\right)10^3 \qquad [5]$$

where ρ = resistivity of the conducting material of the conductor in ohms per mil-foot, as follows:

	25° C	65° C
Hard-drawn copper 97.3% conductivity	10.87	12.54
Hard-drawn aluminum 61% conductivity	17.33	19.99

S = spiraling factor as follows:

Solid conductor	1.00
Stranded conductor	1.02

l = length of line, in miles;

a = cross-sectional area of conducting material of conductor, in circular mils.

The resistance voltage is in phase with the current and is equal to the product of the resistance and the current.

Reactance, X, is expressed in ohms of one conductor and is determined from the equation:

$$X = 2\pi fl\left[8.05 + 74.1\log_{10}\left(\frac{2Ds}{d}\right)\right]10^{-5} \qquad [6]$$

where $\pi = 3.1416$;

f = frequency, in cycles per second;

l = length of line, in miles;

D = spacing of conductors, in inches (for irregular spacing a, b, c, $D = \sqrt[3]{\quad}$

s = stranding factor or ratio of over-all diameter of cable to diameter of equivalent solid conductor, as follows:

Solid conductor	1.00
Standard stranded conductor	1.15 *
Steel-reinforced stranded conductor	1.23 *
Rope-core stranded conductor	1.35 *
Hollow stranded conductor	1.35 *

d = over-all diameter of conductor, in inches.

The reactance voltage is in leading quadrature with the current and is equal to the product of the reactance and the current.

Impedance, Z, is expressed in ohms of one conductor and is determined from the equations:

$$\left.\begin{aligned} \dot{Z} &= R + jX \\ Z &= \sqrt{R^2 + X^2} \end{aligned}\right\} \qquad [7]$$

The impedance voltage leads the current and is equal to the product of the impedance and the current.

Conductance, G, is expressed in mhos of one conductor and is determined from the leakage of current from conductor to conductor, either through the air or over the surfaces of insulators. Where the conductors are properly separated and insulated the conductance is so small at the commercial frequencies and lengths of power lines that it is generally considered that:

$$G = 0 \qquad [8]$$

The conductance current, if it were appreciable, would be in phase with the voltage and equal to the product of the conductance and the voltage.

Susceptance, B, is expressed in mhos of one conductor and is determined from the equation:

$$B = 2\pi fl\left[\frac{3.88}{\log_{10}\left(\frac{2D}{d}\right)}\right]10^{-8} \qquad [9]$$

* These values are approximate. Exact values depend on the design of the cable.

where $\pi = 3.1416$;
f = frequency, in cycles per second;
l = length of line, in miles;
D = spacing of conductors, in inches (for irregular spacing a, b, c, $D = \sqrt[3]{abc}$);
d = over-all diameter of conductor, in inches.

The susceptance current, commonly called the charging current, is in leading quadrature with the voltage and is equal to the product of the susceptance and the voltage.

Admittance, Y, is expressed in mhos of one conductor and is determined from the equation:

$$\left.\begin{aligned} \dot{Y} &= G + jB \\ Y &= \sqrt{G^2 + B^2} \end{aligned}\right\} \qquad [10]$$

The admittance current leads the voltage and is equal to the product of the admittance and the voltage.

The derived constants, A, B, and C, are obtained from the fundamental constants by means of the following equations:

$$\left.\begin{aligned} \dot{A} &= a_1 + ja_2 = \left(1 + \frac{\dot{Y}\dot{Z}}{2} + \frac{\dot{Y}^2\dot{Z}^2}{24} + \frac{\dot{Y}^3\dot{Z}^3}{720} + \frac{\dot{Y}^4\dot{Z}^4}{40320} + \cdots\right) \\ \dot{B} &= b_1 + jb_2 = \dot{Z}\left(1 + \frac{\dot{Y}\dot{Z}}{6} + \frac{\dot{Y}^2\dot{Z}^2}{120} + \frac{\dot{Y}^3\dot{Z}^3}{5040} + \frac{\dot{Y}^4\dot{Z}^4}{362880} + \cdots\right) \\ \dot{C} &= c_1 + jc_2 = \dot{Y}\left(1 + \frac{\dot{Y}\dot{Z}}{6} + \frac{\dot{Y}^2\dot{Z}^2}{120} + \frac{\dot{Y}^3\dot{Z}^3}{5040} + \frac{\dot{Y}^4\dot{Z}^4}{362880} + \cdots\right) \end{aligned}\right\} \qquad [11]$$

These constants take into account the fact that the fundamental constants are actually distributed along the entire length of the line. They are used in the exact performance equations given in later paragraphs. Any degree of accuracy can be obtained by using a sufficient number of terms of the series. It is rarely necessary, however, to use more than three terms.

In the solution of a single-phase or a polyphase circuit, it is assumed that the circuit is symmetrical about its neutral, and, therefore, the solution of one leg of the circuit determines the performance of the entire circuit. Therefore, the constants R, X, G, and B are always given for one wire only and are used as one-wire values in all equations. The kva, voltage, and current may be expressed in terms of one leg also, and the result will then be in terms of one leg. If preferred, however, the 1-wire constants can be used with kva, voltage, and current expressed in terms of the entire circuit, and the results will then be in terms of the entire circuit. This latter method is preferred by many engineers. The two methods are fully defined in the following tabulation:

Method 1. In Terms of One Leg

K = volt-amperes per conductor
= volt-amperes of entire circuit divided by
2 for a 2-wire, single-phase circuit
3 for a 3-wire, 3-phase circuit
4 for a 4-wire, 2-phase circuit;
E = volts from conductor to neutral
= volts between conductors divided by
2 for a 2-wire, single-phase circuit
3 for a 3-wire, 3 phase circuit
2 for a 4-wire, 2-phase circuit;
I = amperes per conductor;
$K = EI$.

Method 2. In Terms of Entire Circuit

K = volt-amperes of the entire circuit;
E = volts between conductors;
I = amperes of the entire circuit
= amperes per conductor multiplied by
1 for a 2-wire, single-phase circuit
$\sqrt{3}$ for a 3-wire, 3-phase circuit
2 for a 4-wire, 2-phase circuit;
$K = EI$.

Conditions of power, voltage, and current are usually fixed at the receiving end by requirements of the load, and then computation is made to determine corresponding conditions at the sending end. Occasionally, however, conditions are fixed at the generating end and computations must be made to obtain the corresponding conditions at the receiving end.

9. Approximate Performance Equations. The performance of a short line, particularly when operated at low frequency, may be determined by approximate equations which neglect the distributed nature of the constants and ignore the susceptance. The derived constants are not needed for this solution. The error is of the order of:

0.1% for a 1200 cycle-mi line
0.5% for a 3100 cycle-mi line
1.0% for a 4200 cycle-mi line

Thus, with an error not exceeding 0.5%, the approximate solution may be used for a 60-cycle, 52-mi line, or for a 25-cycle, 124-mi line.

The equivalent circuit of a short line may be represented as in Fig. 6. The current and voltage vector diagram for the case of lagging power factor at

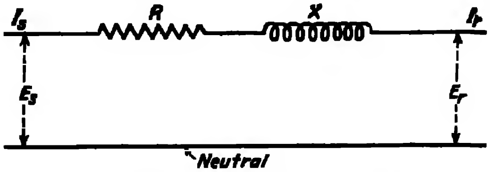


FIG. 6. Short line—equivalent circuit.

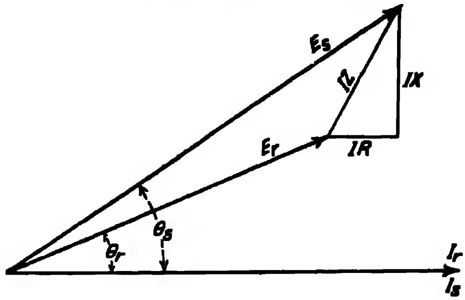


FIG 7. Short line—vector diagram.

both ends of the line is shown in Fig. 7. The figures represent one leg of the circuit, the imaginary neutral having zero resistance and reactance. The notation and equations are as follows:

Notation

R = resistance } fundamental constants as defined in Section 8;
X = reactance }
P = power of entire circuit in watts;
Q = reactive volt-amperes of entire circuit (positive for leading power factor and negative for lagging power factor);
$K = P + jQ$ = volt-amperes of entire circuit;
PF = power factor;
$\theta = \cos^{-1} PF$; $\sin\theta$ is positive when PF is lagging and negative when PF is leading;
E = voltage between conductors, in volts;
I = current of entire circuit, in amperes;
r and s used as subscripts refer to receiving end and sending end respectively.

Equations for Fixed Conditions at Receiving End

$$\left.\begin{aligned}
P_r &= K_r \cos\theta_r = E_r I \cos\theta_r \\
E_s &= \sqrt{(E_r \cos\theta_r + IR)^2 + (E_r \sin\theta_r + IX)^2} \\
I_s &= I_r = I \\
PF_s &= \cos\theta_s = \frac{E_r \cos\theta_r + IR}{E_s} \\
P_s &= E_s I \cos\theta_s = E_r I \cos\theta_r + I^2 R
\end{aligned}\right\} \quad [12]$$

Equations for Fixed Conditions at Sending End

$$\left.\begin{aligned}
P_s &= K_s \cos\theta_s = E_s I \cos\theta_s \\
E_r &= \sqrt{(E_s \cos\theta_s - IR)^2 + (E_s \sin\theta_s - IX)^2} \\
I_r &= I_s = I \\
PF_r &= \cos\theta_r = \frac{E_s \cos\theta_s - IR}{E_r} \\
P_r &= E_r I \cos\theta_r = E_s I \cos\theta_s - I^2 R
\end{aligned}\right\} \quad [13]$$

The use of the above equations is illustrated by the following typical short-line problem:

A transmission line 40 mi long consists of 3 conductors of No. 1/0 B. & S., 106,000 cir mils, hard-drawn stranded copper, over-all diameter 0.373 in., arranged at the vertices of a triangle with sides 80, 85, and 90 in., respectively. Determine the line performance with a frequency of 60 cycles per second and with fixed conditions of 15,000 kw, 80% lagging power factor, 66,000 volts (*a*) at the receiving end, and (*b*) at the sending end.

The line constants are computed from Eqs. 5 and 6 as follows:

$$R = 5.28 \times 10.87 \times 1.02\left(\frac{40}{106{,}000}\right)10^3$$
$$= 22.1$$
$$X = 2 \times 3.1416 \times 60 \times 40\left[8.05 + 74.1\log_{10}\left(\frac{2 \times 84.92 \times 1.15}{0.373}\right)\right]10^{-5}$$
$$= 31.6$$

(*a*) Performance with fixed conditions at the receiving end is computed from Eq. 12 as follows:

$$P_r = 15{,}000{,}000$$
$$\cos\theta_r = 0.8;\ \sin\theta = 0.6$$
$$E_r = 66{,}000$$
$$I = \frac{15{,}000{,}000}{0.8 \times 66{,}000} = 284$$
$$E_s = \sqrt{(66{,}000 \times 0.8 + 284 \times 22.1)^2 + (66{,}000 \times 0.6 + 284 \times 31.6)^2}$$
$$= 76{,}482$$
$$\cos\theta_s = \frac{(66{,}000 \times 0.8) + (284 \times 22.1)}{76{,}482}$$
$$= 0.772$$
$$P_s = 76{,}482 \times 284 \times 0.772$$
$$= 16{,}768{,}500$$

(*b*) Performance with fixed conditions at the sending end is computed from Eq. 13 as follows:

$$P_s = 15{,}000{,}000$$
$$\cos\theta_s = 0.8;\ \sin\theta_s = 0.6$$
$$E_s = 66{,}000$$
$$I = \frac{15{,}000{,}000}{0.8 \times 66{,}000} = 284$$
$$E_r = \sqrt{(66{,}000 \times 0.8 - 284 \times 22.1)^2 + (66{,}000 \times 0.6 - 284 \times 31.6)}$$
$$= 55{,}699$$
$$\cos\theta_r = \frac{(66{,}000 \times 0.8) - (284 \times 22.1)}{55{,}699}$$
$$= 0.835$$
$$P_r = 55{,}699 \times 284 \times 0.835$$
$$= 13{,}208{,}500$$

The calculation should be checked graphically by a simple diagram, as shown in Fig. 7. It should be remembered that, for fixed conditions at the sending end, the vectors IR and IX must be drawn backward, that is, 180 degrees from the positions shown in the figure, as their signs are negative in Eq. 13. For other methods of calculation see Chapter 40, Section 17, Ref. 20.

10. Exact Performance Equations. The performance of long transmission lines, particularly when operated at 60 cycles per second, can be determined accurately only by the use of exact equations which take into account the distributed nature of the constants, including susceptance. The derived

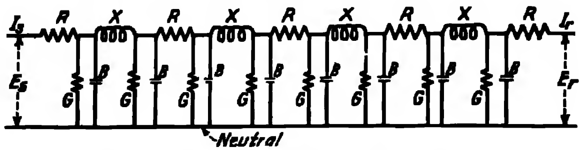


FIG. 8. Long line—equivalent circuit.

constants are required for this solution. The limits of length and frequency beyond which the exact methods are needed to avoid excessive error are given in Section 9.

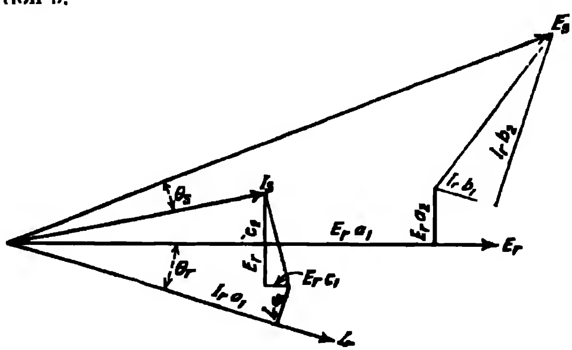


FIG. 9 Long line—vector diagram.

The equivalent circuit of a long line may be represented as in Fig. 8. The voltage and current vector diagram for the case of lagging power factor at both ends of the line is shown in Fig. 9. The notation and equations are as follows:

Notation

R = resistance
X = reactance
G = conductance
B = susceptance

Fundamental constants as defined in Section 8;

$\dot{A} = a_1 + ja_2$
$\dot{B} = b_1 + jb_2$
$\dot{C} = c_1 + jc_2$

Derived constants as defined in Section 8;

P = power of entire circuit, in watts;
Q = reactive volt-amperes (positive for leading power factor and negative for lagging power factor);
$\dot{K} = P + jQ$ = volt-amperes of entire circuit;
PF = power factor;
$\theta = \cos^{-1} PF$;
e_1 = in-phase component of voltage;
e_2 = quadrature component of voltage;
$\dot{E} = e_1 + je_2$ = voltage between conductors in volts. (As E_r is chosen as reference vector, $e_{r1} = E_r$ and $e_{r2} = 0$. e_{s2} is positive when E_s leads E_r and negative when E_s lags E_r);
i_1 = in-phase component of current;
i_2 = quadrature component of current;
$\dot{I} = i_1 + ji_2$ = current of entire circuit in amperes (i_{r2} is positive when I_r leads E_r and negative when I_r lags E_r. i_{s2} is positive when I_s leads E_r and negative when I_s lags E_r);
r and s, used as subscripts, refer to receiving end and sending end, respectively.

Equations for Fixed Conditions at Receiving End

$$\left.\begin{aligned}
P_r &= K_r \cos\theta_r\ E_rI_r \cos\theta_r \\
\dot{E}_s &= \dot{E}_r\dot{A} + \dot{I}_r\dot{B} \\
\dot{I}_s &= \dot{I}_r\dot{A} + \dot{E}_r\dot{C} \\
PF_s &= \cos\theta_s = \frac{e_{s1}i_{s1} + e_{s2}i_{s2}}{E_sI_s} \\
P_s &= E_sI_s\cos\theta_s = e_{s1}i_{s1} + e_{s2}i_{s2}
\end{aligned}\right\} \qquad [14]$$

Equations for Fixed Conditions at Sending End

$$\left.\begin{aligned}
P_s &= K_s \cos\theta_s = E_sI_s \cos\theta_s \\
\dot{E}_r &= \dot{E}_s\dot{A} - \dot{I}_s\dot{B} \\
\dot{I}_r &= \dot{I}_s\dot{A} - \dot{E}_s\dot{C} \\
PF_r &= \cos\theta_r = \frac{e_{r1}i_{r1} + e_{r2}i_{r2}}{E_rI_r} \\
P_r &= E_rI_r\cos\theta_r = e_{r1}i_{r1} + e_{r2}i_{r2}
\end{aligned}\right\} \qquad [15]$$

The use of the above equations is illustrated by the following typical long-transmission-line problem:

A transmission line 200 mi long consists of 3 conductors of 605,000 cir mils aluminum cable, steel-reinforced, over-all diameter 0.953 in., arranged in a plane with spacing 14, 14, and 28 ft respectively. Determine the line performance with a frequency of 60 cycles per second, and with fixed conditions of 40,000 kw, 90% lagging power factor, 132,000 volts (a) at the receiving end and (b) at the sending end.

The line constants are computed from Eqs. 5, 6, 7, 8, 9, 10, and 11 as follows:

$$R = 5.28 \times 17.33 \times 1.02 \left(\frac{200}{605{,}000}\right) 10^3$$
$$= 30.854;$$
$$X = 2\times 3.1416\times 60\times 200 \left[8.05 + 74.1 \log_{10}\left(\frac{2\times 211.68\times 1.23}{0.953}\right)\right] 10^{-5}$$
$$= 159.015;$$
$$\dot{Z} = 30.854 + j159.015;$$
$$G = 0;$$
$$B = 2 \times 3.1416 \times 60 \times 200 \left[\frac{3.88}{\log_{10}\left(\frac{2 \times 211.68}{0.953}\right)}\right] 10^{-8}$$
$$= 0.0011049;$$
$$\dot{Y} = 0 + j0.0011049;$$
$$\dot{A} = 0.91338 + j0.01655;$$
$$A = 0.91354;$$
$$\dot{B} = 29.07041 + j154.5701;$$
$$B = 157.28;$$
$$\dot{C} = -0.000006 + j0.001073;$$
$$C = 0.001073.$$

(*a*) Performance with fixed conditions at the receiving end is computed from Eq. 14 as follows:

$$P_r = 40{,}000{,}000;$$
$$\cos\theta_r = 0.9; \sin\theta_r = 0.436;$$
$$E_r = 132{,}000;$$
$$e_{r1} = 132{,}000;$$
$$e_{r2} = 0;$$
$$\dot{E}_r = 132{,}000;$$
$$I_r = \frac{40{,}000{,}000}{0.9 \times 132{,}000} = 336.70;$$
$$i_{r1} = 0.9 \times 336.7 = 303.03;$$
$$i_{r2} = 0.436 \times 336.7 = 146.80;$$
$$\dot{I}_r = 303.03 - j146.80;$$
$$\dot{E}_s = (132{,}000)(0.91338 + j0.01655) + (303.03 - j146.80)(29.07041 + j154.5701)$$
$$= 152{,}066 + j44{,}756;$$
$$E_s = 158{,}516;$$
$$\dot{I}_s = (303.03 - j146.80)(0.91338 + j0.01655) + (132{,}000)(-0.000006 + j0.001073)$$
$$= 278.4 + j12.57;$$
$$I_s = 278.7;$$
$$\cos\theta_s = \frac{(152{,}066 \times 278.4) + (44{,}756 \times 12.57)}{158{,}516 \times 278.7}$$
$$= 0.971 \text{ lagging};$$
$$P_s = (152{,}066 \times 278.4) + (44{,}756 \times 12.57)$$
$$= 42{,}898{,}000.$$

(*b*) Performance with fixed conditions at the sending end is computed from Eq. 15 as follows:

$$P_s = 40{,}000{,}000;$$
$$\cos\theta_s = 0.9;\ \sin\theta_s = 0.436;$$
$$E_s = 132{,}000;$$
$$e_{s1} = 132{,}000;$$
$$e_{s2} = 0;$$
$$\dot{E}_s = 132{,}000;$$
$$I_s = \frac{40{,}000{,}000}{0.9 \times 132{,}000} = 336.70;$$
$$i_{s1} = 0.9 \times 336.70 = 303.03;$$
$$i_{s2} = 0.436 \times 336.70 = 146.80;$$
$$\dot{I}_s = 303.03 - j146.80;$$
$$\dot{E}_r = (132{,}000)(0.91338 + j0.01655) - (303.03 - j146.80)(29.07041 + j154.5701)$$
$$= 89{,}066 - j40{,}387;$$
$$E_r = 97{,}795;$$
$$\dot{I}_r = (303.03 - j146.80)(0.91338 + j0.01655) - (132{,}000)(-0.000006 + j0.001073)$$
$$= 280.0 - j270.7;$$
$$I_r = 389.4;$$
$$\cos\theta_r = \frac{(89{,}066 \times 280) + (40{,}387 \times 270.7)}{97{,}795 \times 389.4}$$
$$= 0.942 \text{ lagging};$$
$$P_r = (89{,}066 \times 278.4) + (40{,}387 \times 270.7)$$
$$= 35{,}871{,}000.$$

The calculation should be checked graphically by a simple diagram as shown in Fig. 9. It should be remembered that for fixed conditions at the sending end the vectors $\dot{I}\dot{B}$ and $\dot{E}\dot{C}$ must be drawn backward, that is, 180 degrees from the positions shown in the figure, as their signs are negative in Eq. 15.

For other methods of calculation, see Chapter 40, Section 17, Ref. 20.

11. Transformer Constants. The transformers at each end of the transmission line must be considered in determining the performance of the system as a whole. The constants of a bank of transformers may be expressed in the same terms as the fundamental constants of the line, as follows:

$$\left.\begin{aligned} R &= \frac{rE^2}{100K} \\ X &= \frac{xE^2}{100K} \\ \dot{Z} &= R + jX \\ G &= \frac{gK}{100E^2} \\ B &= \frac{bK}{100E^2} \\ \dot{Y} &= G - jB \end{aligned}\right\} \qquad [16]$$

where R = resistance per conductor, in ohms;
X = reactance per conductor, in ohms;
G = conductance per conductor, in mhos;
B = susceptance per conductor, in mhos;
r = resistance volts, in percentage of rated volts;
x = reactance volts, in percentage of rated volts;
g = iron-loss current, in percentage of rated current;
b = magnetizing current, in percentage of rated current;
E = rated voltage between conductors;
K = rated capacity, in volt-amperes.

Approximate values of r, x, g, and b suitable for preliminary transmission-system calculations are given in Table 1. As these characteristics vary considerably with the transformer design, actual values for final calculations should be obtained from the transformer manufacturer.

TABLE 1

TYPICAL TRANSFORMER CHARACTERISTICS
60-cycle, Single-phase

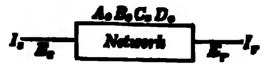
Rated Kva	Per Cent Resistance	Per Cent Reactance	Per Cent Iron Loss	Per Cent Magnetizing Current
W. C. 110 kv				
500	1.5	8.0	1.0	8.0
1,000	1.4	9.0	0.8	7.3
2,000	1 2	9.0	0.7	6.2
5,000	1.0	9.5	0.57	5.0
10,000	0.85	9.5	0.48	4.3
20,000	0.75	10.0	0.42	3.6
30,000	0.70	10.0	0.38	3 3
W. C. 220 kv				
5,000	1 2	10 5	0.81	6.9
10,000	1.0	11.0	0.67	5.9
20 000	0.86	12.5	0.52	4.7
30,000	0.80	13.0	0.44	4.2
S. C. 110 kv				
1,000	0.82	10.0	0.92	9.0
2,000	0 75	10.0	0.67	5.0
5,000	0.63	10.0	0.45	3.0
10,000	0.52	10.0	0.36	2.4
20,000	0.42	10.0	0.31	2.1
30,000	0.39	10.0	0.30	2.0

12. Networks. A convenient method of determining the performance of a network consisting of transmission lines, transformers, and other impedances and admittances was published by Evans and Sels in the *Electric Journal*, for August 1921, p. 356. The method consists of combining the constants of the various elements of the system so as to give four general circuit con-

TABLE 2

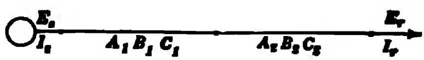
Circuit Constants for Elementary Networks

General Circuit Equations:



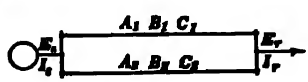
$$E_s = E_rA_0 + I_rB_0$$
$$I_s = I_rD_0 + E_rC_0$$
$$E_r = E_sD_0 - I_sB_0$$
$$I_r = I_sA_0 - E_sC_0$$

Two Transmission Lines in Series:



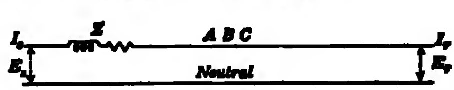
$$A_0 = a_1 + ja_2 = A_1A_2 + C_1B_2$$
$$B_0 = b_1 + jb_2 = B_1A_2 + D_1B_2$$
$$C_0 = c_1 + jc_2 = A_1C_2 + C_1D_2$$
$$D_0 = d_1 + jd_2 = B_1C_2 + D_1D_2$$

Two Transmission Lines in Multiple:



$$A_0 = a_1 + ja_2 = \frac{A_1B_2 + B_1A_2}{B_1 + B_2}$$
$$B_0 = b_1 + jb_2 = \frac{B_1B_2}{B_1 + B_2}$$
$$C_0 = c_1 + jc_2 = C_1 + C_2 + \frac{(A_1 - A_2)(D_2 - D_1)}{B_1 + B_2}$$
$$D_0 = d_1 + jd_2 = \frac{B_1D_2 + D_1B_2}{B_1 + B_2}$$

Transmission Line with Series Impedance at Sending End:

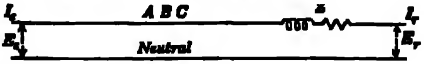


$$A_0 = a_1 + ja_2 = A + CZ$$
$$B_0 = b_1 + jb_2 = B + AZ$$
$$C_0 = c_1 + jc_2 = C$$
$$D_0 = d_1 + jd_2 = A$$

TABLE 2—*Continued*

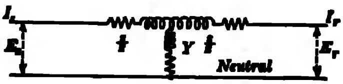
CIRCUIT CONSTANTS FOR ELEMENTARY NETWORKS

Transmission Line with Series Impedance at Receiving End:



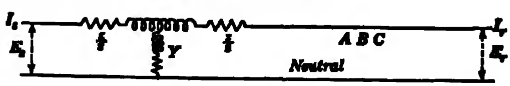
$$A_0 = a_1 + ja_2 = A$$
$$B_0 = b_1 + jb_2 = B + AZ$$
$$C_0 = c_1 + jc_2 = C$$
$$D_0 = d_1 + jd_2 = A + CZ$$

One Bank of Transformers:



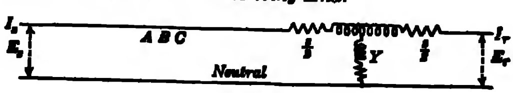
$$A_0 = a_1 + ja_2 = 1 + \frac{ZY}{2}$$
$$B_0 = b_1 + jb_2 = Z\left(1 + \frac{ZY}{4}\right)$$
$$C_0 = c_1 + jc_2 = Y$$
$$D_0 = d_1 + jd_2 = 1 + \frac{ZY}{2}$$

Transmission Line with Transformers at Sending End



$$A_0 = a_1 + ja_2 = A\left(1 + \frac{ZY}{2}\right) + CZ\left(1 + \frac{ZY}{4}\right)$$
$$B_0 = b_1 + jb_2 = B\left(1 + \frac{ZY}{2}\right) + AZ\left(1 + \frac{ZY}{4}\right)$$
$$C_0 = c_1 + jc_2 = C\left(1 + \frac{ZY}{2}\right) + AY$$
$$D_0 = d_1 + jd_2 = A\left(1 + \frac{ZY}{2}\right) + BY$$

Transmission Line with Transformers at Receiving End:

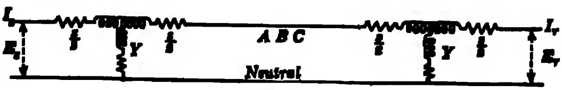


$$A_0 = a_1 + ja = A\left(1 + \frac{ZY}{2}\right) + BY$$
$$B_0 = b_1 + jb_2 = B\left(1 + \frac{ZY}{2}\right) + AZ\left(1 + \frac{ZY}{4}\right)$$
$$C_0 = c_1 + jc_2 = C\left(1 + \frac{ZY}{2}\right) + AY$$
$$D_0 = d_1 + jd_2 = A\left(1 + \frac{ZY}{2}\right) + CZ\left(1 + \frac{ZY}{4}\right)$$

TABLE 2 *Continued*

CIRCUIT CONSTANTS FOR ELEMENTARY NETWORKS

Transmission Line with Transformers at Each End:



$$A_0 = a_1 + ja_2 = A\left[\left(1 + \frac{Z_rY_r}{2}\right)\left(1 + \frac{Z_sY_s}{2}\right) + Z_sY_r\left(1 + \frac{Z_sY_s}{4}\right)\right] + BY_r\left(1 + \frac{Z_sY_s}{2}\right) + CZ_s\left(1 + \frac{Z_rY_r}{2}\right)\left(1 + \frac{Z_sY_s}{4}\right)$$

$$B_0 = b_1 + jb_2 = B\left(1 + \frac{Z_rY_r}{2}\right)\left(1 + \frac{Z_sY_s}{2}\right) + A\left[Z_r\left(1 + \frac{Z_rY_r}{4}\right)\left(1 + \frac{Z_sY_s}{2}\right) + Z_s\left(1 + \frac{Z_rY_r}{2}\right)\left(1 + \frac{Z_sY_s}{4}\right)\right] + CZ_sZ_r\left(1 + \frac{Z_rY_r}{4}\right)\left(1 + \frac{Z_sY_s}{4}\right)$$

$$C_0 = c_1 + jc_2 = C\left(1 + \frac{Z_rY_r}{2}\right)\left(1 + \frac{Z_sY_s}{2}\right) + A\left[Y_r\left(1 + \frac{Z_sY_s}{2}\right) + Y_s\left(1 + \frac{Z_rY_r}{2}\right)\right] + BY_rY_s$$

$$D_0 = d_1 + jd_2 = A\left[\left(1 + \frac{Z_rY_r}{2}\right)\left(1 + \frac{Z_sY_s}{2}\right) + Z_rY_s\left(1 + \frac{Z_rY_r}{4}\right)\right] + BY_s\left(1 + \frac{Z_rY_r}{2}\right) + CZ_r\left(1 + \frac{Z_rY_r}{4}\right)\left(1 + \frac{Z_sY_s}{2}\right)$$

stants, A_0, B_0, C_0, and D_0, which may be used in the usual exact transmission-line equations The more important elements with their equations are given in Table 2 For a system symmetrical about its center, such as a simple transmission line, the constant D_0 is equal to A_0

13. Regulation and Power Diagrams. Graphical methods of determining the performance of transmission systems are exceedingly useful because of their simplicity and the variety of solutions that may be obtained from a single diagram. It is always desirable to check at least one point on the diagram by the analytical method, to prove its accuracy. Many diagrams and charts have been devised, each having particular merit for certain kinds of analyses. The regulation and power diagrams presented in the following paragraphs are particularly useful for the usual transmission problems.

The regulation diagram is shown in Fig 10. It is reproduced, with slight modifications, from *University of Washington Engineering Experimental Station Bulletin* 32, through the courtesy of the authors, F. K. Kirsten and E. A. Loew. The accuracy of the diagram is limited only by the degree of care exercised in constructing and reading it, as it is based on the exact performance equations. The diagram may be constructed to show the regulation

of any form of network by first computing the four general circuit constants, A_0, B_0, C_0, and D_0, as explained in Section 12. The subsequent operations in constructing and reading the diagram for fixed conditions at the receiving end are as follows:

1. Using a sheet of plain paper, lay off E_r horizontally from O to a convenient voltage scale.
2. Construct angle $\tan^{-1}(a_2/a_1)$.
3. Lay off E_rA_0 to the voltage scale.

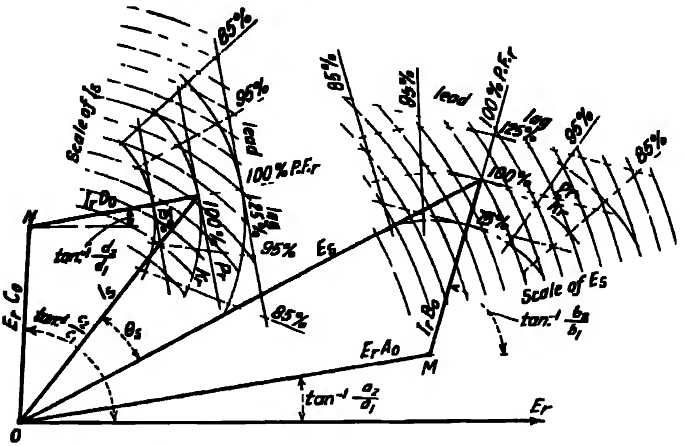


FIG. 10. Regulation diagram for fixed conditions at receiving end.

4. Construct angle $\tan^{-1}(b_2/b_1)$.
5. Lay off I_rB_0 to the voltage scale, using the value of I_r corresponding to full load, unity power factor.
6. Draw K_r circles about center M, using I_rB_0 for radius of 100% circle and others in proportion.
7. Draw P_r lines perpendicular to I_rB_0 through points of intersection with K_r circles.
8. Draw PF_r lines radially from M, by laying off sines or tangents as preferred.
9. Draw E_s circles about center O, using the voltage scale.
10. Construct angle $\tan^{-1}(c_2/c_1)$.
11. Lay off E_rC_0 to a convenient current scale.
12. Construct angle $\tan^{-1}(d_2/d_1)$.
13. Lay off I_rD_0 to the current scale, using the value of I_r corresponding to full load, unity power factor.
14. Draw K_r circles, about center N, using I_rD_0 for radius of 100% circle and others in proportion.
15. Draw P_r lines perpendicular to I_rD_0 through points of intersection with K_r circles.
16. Draw PF_r lines radially from N, by laying off sines or tangents as preferred.
17. Draw I_s circles about center O, using the current scale.
18. To use the diagram, select any load P_r, and power factor PF_r, and read E_s, I_s, and θ_s from which I and efficiency can be readily obtained by slide rule.

The power diagram is shown in Fig 11. It is reproduced, with slight modifications, from the diagram developed by Evans and Sels in the *Electric*

Journal December 1921, p 530 The accuracy of the diagram is limited only by the degree of care exercised in constructing and reading it, as it is based

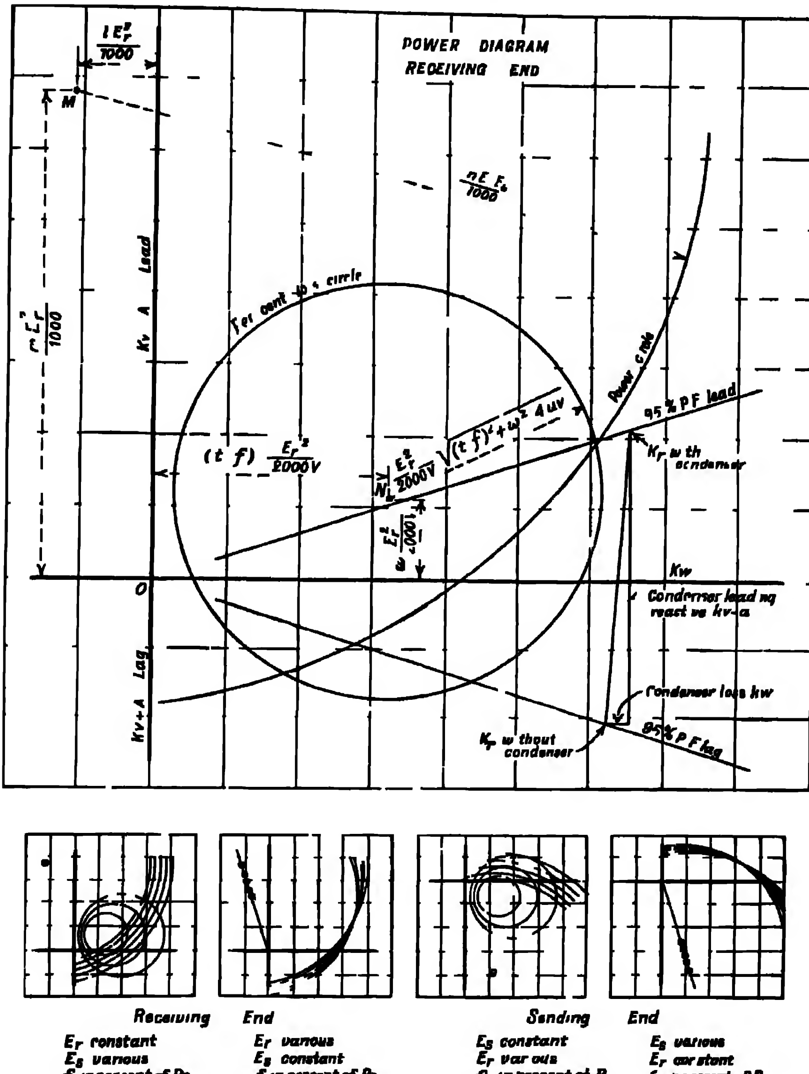


Fig 11 Receiver end power diagram

on the exact performance equations The diagram may be constructed to show the performance of any form of network by first computing the four general circuit constants, as explained in Section 12 It may be constructed

in any one of four general forms shown at the bottom of Fig. 11. In order to show the particular performance required by the analysis. After the general circuit constants, A_0, B_0, C_0, and D_0, have been computed, the subsequent operations in constructing and reading the diagram are as follows:

1. Using a sheet of cross-section paper, lay off suitable scales of kw, kva lead, and kva lag along the axes passing through O. The proper quadrants in which to locate the centers of the circles are determined by an inspection of the signs as explained below. The equation of a circle with center at point $+a$, $+b$, in the first quadrant is:

$$(x - a)^2 + (y - b)^2 = C^2 \tag{17}$$

2. Draw PF lines radially from O by laying off sines or tangents as preferred.
3. Locate centers M, and draw several power circles, using the power-circle equation given below and varying E_r or E_s as desired.
4. Locate centers N and draw several per cent loss circles, using the per-cent-loss-circle equation as given below and varying f as desired.
5. From the completed diagram, with fixed power, power factor, and voltage at one end of the system, the voltage at the other end and the per cent power loss may be read directly. The effect of a condenser at the receiving end can be obtained from the condenser triangle as shown on the diagram.

The notation and equations for power at the receiving end are as follows:

Notation

P_r = total 3-phase power component at receiving end in kilowatts;
Q_r = total 3-phase reactive component at receiving end in kilovolt-amperes (Q_r is positive for leading power factor and negative for lagging power factor);
$K_r = P_r + jQ_r$ = total 3-phase volt-amperes at receiving end;
E_r = line voltage at receiving end in volts;
E_s = line voltage at sending end in volts;

$$l = \frac{a_1 b_1 + a_2 b_2}{b_1^2 + b_2^2};$$

$$m = \frac{a_1 b_2 - a_2 b_1}{b_1^2 + b_2^2};$$

$$n = \frac{1}{\sqrt{b_1^2 + b_2^2}};$$

$t = (a_1 b_1 + b_1 c_1 + a_2 d_2 + b_2 c_2 - 1)$;
$u = (a_1 c_1 + a_2 c_2)$;
$v = (b_1 d_1 + b_2 d_2)$;
$w = (a_1 d_2 + b_2 c_1 - a_2 d_1 - b_1 c_2)$;
f = power loss in per cent of power at receiving end.

Power-circle Equation

$$\left(P_r + \frac{lE_r^2}{1000}\right)^2 + \left(Q_r - \frac{mE_r^2}{1000}\right)^2 = \left(\frac{nE_rE_s}{1000}\right)^2 \tag{18}$$

Constants l, m, and n are always positive, and therefore the center of the circle is in the second quadrant.

Per-cent-loss-circle Equation

$$\left[P_r + (t - f)\frac{E_r^2}{2000v}\right]^2 + \left[Q_r - w\frac{E_r^2}{2000v}\right]^2$$
$$= \left[\frac{Er^2}{2000v}\sqrt{(t - f)^2 + w^2 - 4uv}\right]^2 \quad [19]$$

Constants u and v are always positive. Constants t and w may be positive or negative, depending on the system characteristics, but, as f is always larger than t, $(t - f)$ is always negative. The center of the circle is, therefore, in the first quadrant if w is positive and in the fourth quadrant if w is negative.

Equation for Point of Maximum Efficiency

$$f = \pm \sqrt{4uv - w^2} + t \quad [20]$$

The notation and equations for power at the sending end are as follows:

Notation

P_s = total 3-phase power component at sending end, in kilowatts;
Q_s = total 3-phase reactive component at sending end, in kilovolt-amperes (Q_s is positive for leading power factor and negative for lagging power factor);
$K_s = P_s + jQ_s$ = total 3-phase volt-amperes at sending end;
E_s = line voltage at sending end, in volts;
E_r = line voltage at receiving end, in volts;
$l' = \dfrac{d_1b_1 + d_2b_2}{b_1^2 + b_2^2}$;
$m' = \dfrac{d_1b_2 - d_2b_1}{b_1^2 + b^2}$;
$n = \dfrac{1}{\sqrt{b_1^2 + b_2^2}}$;
$t = (a_1d_1 + b_1c_1 + a_2d_2 + b_2c_2 - 1)$;
$u' = (c_1d_1 + c_2d_2)$;
$v' = (a_1d_1 + a_2d_2)$;
$w' = (a_2d_2 + b_2c_2 - a_1d_2 - b_1c_2$:
f' = power loss in per cent of power at sending end.

Power-circle Equation

$$\left(P_s - \frac{l'E_s^2}{1000}\right)^2 + \left(Q_s + \frac{m'E_s^2}{1000}\right)^2 = \left(\frac{nE_sE_r}{1000}\right)^2 \quad [21]$$

Constants l', m', and n are always positive, and therefore the center of the circle is in the fourth quadrant.

Per-cent-loss-circle Equation

$$\left[P_s - (t+f')\frac{E_s^2}{2000v'}\right]^2 + \left[Q_s + w'\frac{E_s^2}{2000v'}\right]^2 = \left[\frac{E_s^2}{2000v'}\sqrt{(t+f')^2 + w'^2 - 4u'v'}\right]^2 \quad [22]$$

Constants u' and v' are always positive. Constants t and w' may be positive or negative, depending on the system characteristics, but, as f' is always larger than t, $(t+f')$ is always positive. The center of the circle is, therefore, in the first quadrant if w' is negative and in the fourth quadrant if w' is positive.

Equation for Point of Maximum Efficiency

$$f' = \pm\sqrt{4u'v' - w'^2} - t \quad [23]$$

14. Structural Features. Choice of the type of construction to be adopted for a transmission line should be based on results of studies which not only consider first costs and costs of maintenance and depreciation but which also give due consideration to losses from interruptions to service. Such interruptions are generally brought about by mechanical or electrical failure of some part of the line when unusual conditions subject it to loads in excess of those which it has been designed to withstand. Extremely severe loads from wind or ice or a combination of both may occur only rarely, but line failures meaning serious interruptions to service may result if sufficient strength to carry these loads has not been provided. Interruptions due to line flashovers caused by lightning may or may not be serious, but, if the frequency of their occurrence is to be controlled, consideration must be given to the type and extent of protection to be provided. It therefore follows that the desired reliability of a line must be considered along with general climatic conditions and local conditions prevailing on the whole or any part of the line.

15. Conductor Size and Material. Before proceeding with the design of supporting structures, their dimensions and the loads that they must sustain should be determined. This involves determination of the characteristics of the conductors. If, after pursuing the economic considerations outlined in Section 7, it is found that conductors of different sizes, materials, or construction are satisfactory from the standpoint of electrical service, a study should be made to determine the conductor to be used for maximum economy in total line cost. Conductors having high tensile strength can be strung with less relative sag than those having lower strength, and a corresponding increase in spacing of structures of the same height can be made. Other things

being equal, the line with the longer spans will be more economical because of the saving in insulators and reduction in maintenance cost.

The total cost per mile of line including conductors, insulators, structures, and foundations should be estimated for each conductor under consideration. Care should be taken that the most economical spacing of structures and the most economical conductor tension are used in each case, as the choice may be different for different conductors. Figure 12 illustrates a convenient method of finding the most economical spacing of structures for a given conductor and conductor tension. The most economical conductor tension may be determined by the preparation of such charts for varying tensions.

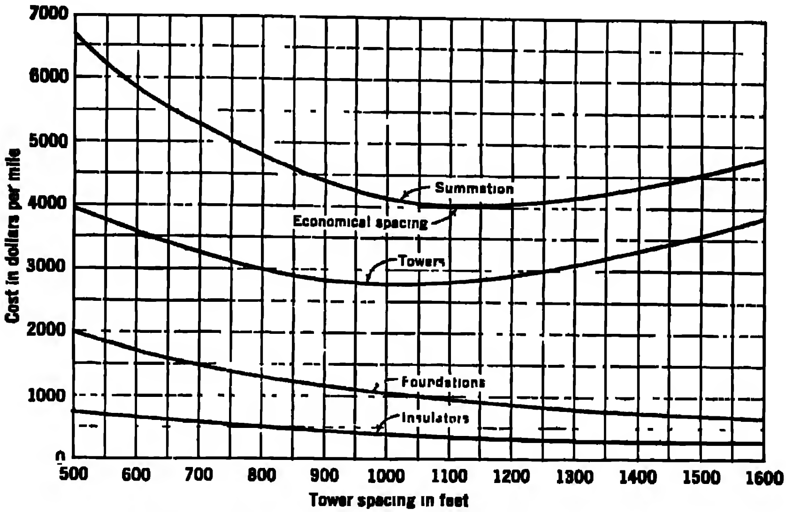


FIG. 12. Determination of economical tower spacing.

In determining economical structure spacing and conductor tension it is not necessary to include items of cost that do not vary, but comparisons of line costs to determine the most economical conductor should of course include the conductor costs.

Figure 13 shows the results of studies made to determine the most economical conductor tension and tower spacing for a 154-kv single-circuit steel tower line built in the heavy loading district with 636,000-cir-mil A.C.S.R. conductors and two 7/16-in. high-strength galvanized-steel ground wires. It shows that in this particular case, with the design assumptions and factors of safety used, the most economical conductor tension under maximum assumed loading for average spans not exceeding about 1250 ft was 9000 lb. It also shows that for spans from about 1250 ft to about 1500 ft the line cost is about the same for limiting conductor tensions between 9000 and 12,000 lb and that for economy the spans should be as long as practical considerations

warrant These figures are given only for illustration and should be used only as a guide in making similar studies for other conductors.

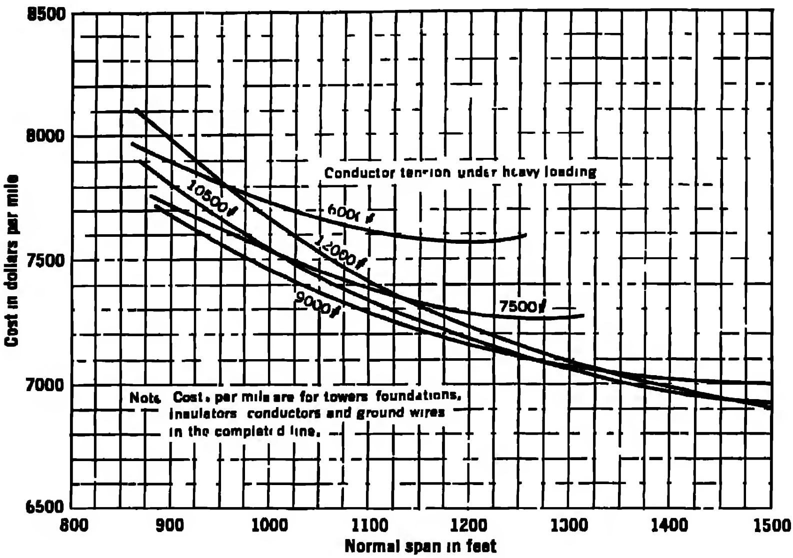


FIG 13 Determination of economical conductor tension

16. Conductor Loading. The heights of supporting structures and the loads to be sustained by them are dependent on the conductor sag and loading

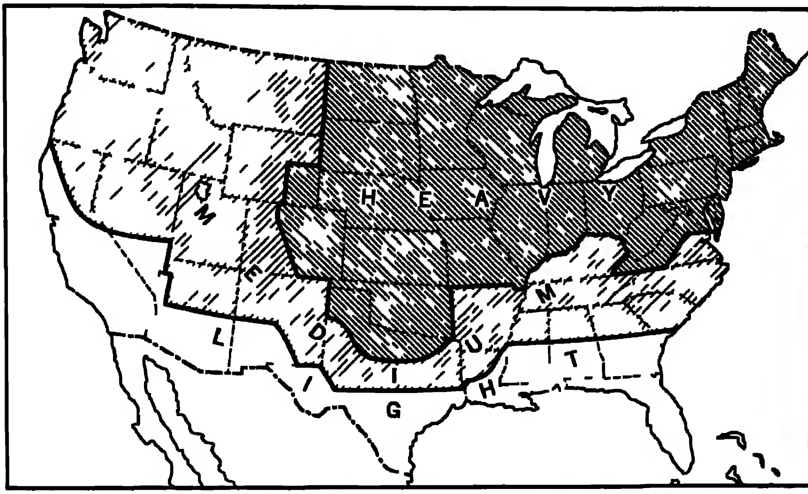
TABLE 3

CONDUCTOR LOADING DATA *

	Loading District		
	Heavy	Medium	Light
Radial thickness of ice, in inches	0.50	0 25	0
Horizontal wind pressure, in pounds per square foot	4	4	9
Temperature, in degrees Fahrenheit	0	+15	+30
Constant to be added to the resultant, in pounds per foot:			
For bare conductors of copper, steel, copper alloy, copper-covered steel, and combinations thereof	0.29	0.19	0 05
For bare conductors of aluminum (with or without steel reinforcement)	0.31	0.22	0 05
For weatherproof and similar covered conductors (all materials)	0 31	0 22	0 05

* *National Electric Safety Code Handbook H*-32 of the National Bureau of Standards, 1941.

Loadings recommended by the National Bureau of Standards for various parts of the United States are shown in Fig 14 and Table 3 These loadings on conductors are assumed to be the resultant loadings per foot equivalent to the vertical loads per foot of conductor, ice-covered where specified, combined with the transverse loads per foot due to a transverse horizontal wind pressure upon the projected area of the conductor, ice-covered where specified



General loading map showing territorial division of the United States with reference to loading of overhead lines

Heavy loading
Medium loading
Light loading
For ice, wind, and temperature values see tabulation

FIG 14 Loading districts for mechanical loading of overhead lines (From *National Electrical Safety Code Handbook* H-32 of the National Bureau of Standards 1941)

To the equivalent resultant loading, the specified constants are to be added for conductor sag and tension calculations

The loadings specified for the loading districts should be considered as minimum and used only as a guide in determining the proper loading, which should give consideration to the importance of service and amount of interruption permissible Lines located in areas where especially severe wind and sleet storms are encountered should be designed for a heavier loading if interruptions to service are to be minimized A wind pressure of 11 lb per sq ft on the projected area of wires coated with a radial thickness of ½ in of ice has been used in the design of some important lines where past experience shows that very heavy sleet formation occurs frequently

Wind pressures on projected areas of cylindrical surfaces of wires are less than on flat surfaces. Several formulas have been devised for the pressure on a wire in terms of the wind velocity. Of these, Bruck's formula for wind pressure on cylindrical surfaces, $P = 0.0025V^2$, is generally accepted for span calculations. In this formula P is the pressure in pounds per square foot of projected area and V is the actual wind velocity in miles per hour.

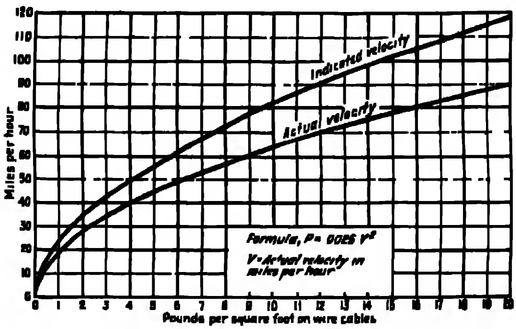


FIG. 15. Wind velocity and pressure.

The relation between actual and indicated wind velocity, and pressures for actual wind velocities up to about 90 mi per hr, are shown in Fig. 15. The relation between actual and indicated wind velocity is not entirely definite, and correction factors should be obtained from the U. S. Weather Bureau with any wind-velocity data. For further discussion of the subject of line loading, see "Mechanical Characteristics of Transmission Lines," by L. E. Imlay, *Electrical Journal*, January 1925.

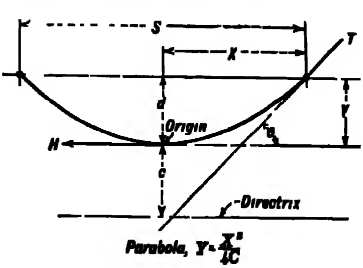


FIG. 16. Parabola.

17. Conductor Sag. A cable of uniform cross-section and material, perfectly flexible but inelastic, suspended at two points in the same horizontal plane and subjected only to its own weight, assumes the form of the common catenary. If the cable is of elastic material, it assumes the form of the elastic catenary. The maximum stress in the cable is at the point of support, and the minimum stress is at the lowest point. The stress at any point in the cable has two components: a horizontal component, which is uniform throughout the length of the cable; and a vertical component, which varies along each half of its length. The vertical component of the tension at any point is equal to one-half the weight of the cable in the span, less the weight of the cable between the point and the nearest point of support. The horizontal component of the tension at any point is equal to the minimum tension in the cable.

If it is assumed that the weight of the cable is distributed uniformly along a horizontal line instead of along the length of the cable, the equation for the curve assumed by the cable will be that of a parabola. There are, therefore, two general methods of calculating sags, the one being based on the elastic catenary, the other on the parabola. The results of the two methods of calculation will be almost identical when the sag is small, but the error due to the parabolic assumption becomes greater as the sag increases. For all practical purposes, the error is negligible for spans up to 1000 ft and can generally be disregarded for spans up to 1500 ft. For spans up to this length, with wires so strung that the sag will not exceed one-tenth of the span, the error will be less than 3%.

The following general equations are derived from the parabola (see Fig. 16):

Sag, in feet, at any point $$d' = \frac{WX^2}{2H} \qquad [24]$$

Maximum sag, in feet $$d = \frac{WS^2}{8H} \qquad [25]$$

Horizontal stress, in pounds $$H = \frac{WS^2}{8d} \qquad [26]$$

Vertical stress, in pounds $$V = \frac{W(3S^2 + 8d^2)}{6S} \qquad [27]$$

Maximum stress, in pounds $$T = \frac{WS}{8d}\sqrt{S^2 + 16d^2} \qquad [28]$$

Length of cable, in feet $$L = S + \frac{8d^2}{3S} \qquad [29]$$

Maximum sag, in feet $$d = \sqrt{\frac{3S(L - S)}{8}} \qquad [30]$$

Change in length of cable, in feet $$e = \frac{TL}{AE} = tL\theta \qquad [31]$$

where W = weight, in pounds per foot of cable and load;
A = area of cross-section of cable, in square inches;
E = modulus of elasticity of cable;
θ = coefficient of expansion for cable;
X = horizontal distance, in feet, from any point in span to the nearest support;
S = length of span, in feet;
t = change of temperature, in degrees Fahrenheit.

18. Stress-deflection Curves. As the cable is subject to changes in temperature, four variables are involved in the solution of the sag problem for

any given span they are length, tension, temperature, and sag, and all are closely interrelated. For example, a change in temperature causes a change in length, this causes a change in sag, which in turn changes the length and tension still further.

It is evident that a mathematical solution to find the sags and tensions for various conditions of loading and for changing temperatures would be very complicated. A combination of mathematical and graphical solutions very much simplifies the problem. If two sets of curves for a given span are drawn on the same sheet, one set showing the relation between sag and tension under various loadings and the other showing the relation between sag and tension at various temperatures, their intersections will show sags and tensions for given loadings and at the different temperatures assumed. Such curves are known as stress-deflection curves and are illustrated in Fig. 17.

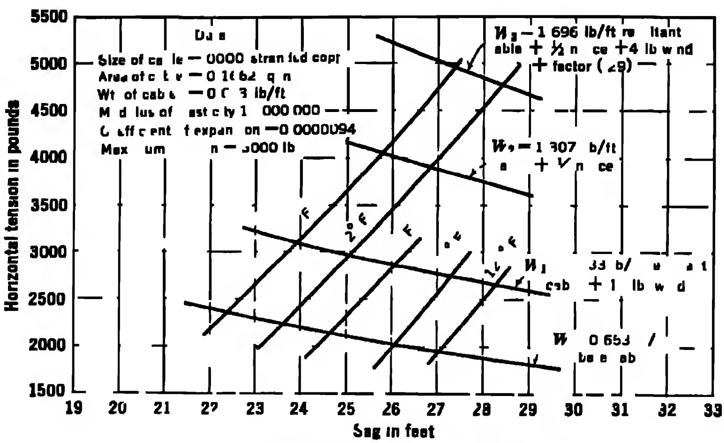


Fig. 17. Stress-deflection curves, 800 ft span.

For convenience, the two systems of curves will be defined as follows:

(*a*) Sag-tension curves—showing the relation between sag and tension for a given span and loading.

(*b*) Pull-up curves—showing the relation between sag and tension for a given span and temperature.

The pull-up curve for any given span is a sag-tension curve at constant temperature and varying load, and it is given this name to distinguish it from the sag-tension curve with a constant load. The term pull up is derived from the method of procedure used in its construction, which consists of gradually reducing the load on the cable at a constant temperature, thereby decreasing both the sag and tension, and pulling up the cable.

The method of preparing these curves by use of the parabola equations is explained by the following problem.

Determine the maximum sag and side swing for a No. 4/0 bare, stranded, hard-drawn copper conductor for a span of 800 ft, the maximum tension in the conductor not to exceed 5000 lb, the assumed loading to be ½ in. radial thickness of ice plus a wind pressure of 4 lb per sq ft of projected area of the ice-covered cable, or a wind pressure of 15 lb per sq ft of projected area of the bare cable, and the assumed temperature range to be from zero to 120° F.

S = span = 800 ft;
T = actual tension in cable (5000 lb maximum);
H = horizontal stress, in pounds, at low point of cable;
A = cross-sectional area of cable = 0.1662 sq in.;
E = modulus of elasticity of cable = 15,000,000;
θ = coefficient of linear expansion for cable = 0.0000094;
W = 0.653 lb per lin ft (weight of bare cable);
W_1 = 0.933 lb per lin ft (resultant of cable weight and wind pressure);
W_2 = 1.307 lb per lin ft (weight of cable plus ice);
W_3 = 1.406 + factor 0.29 = 1.696 lb per lin ft (resultant of cable plus ice weight and wind pressure);
EA = 2,493,000;
t = temperature change in degrees Fahrenheit.

19. Sag-tension Curves. Using Eq. 25, compute the sags for varying horizontal tensions for each of the loadings, W, W_1, W_2, and W_3 as follows:

Loading W	Loading W_1	Loading W_2	Loading W_3
$d = \frac{0.653 \times 800^2}{8H}$	$d = \frac{0.933 \times 800^2}{8H}$	$d = \frac{1.307 \times 800^2}{8H}$	$d = \frac{1.696 \times 800^2}{8H}$
$= \frac{52{,}240}{H}$	$= \frac{76{,}400}{H}$	$= \frac{104{,}560}{H}$	$= \frac{135{,}680}{H}$

W		W_1		W_2		W_3	
H	d	H	d	H	d	H	d
2600	20.1	3500	21.3	4200	24.9	5200	26.1
2400	21.7	3200	23.3	4000	26.1	5000	27.1
2200	23.7	3000	24.9	3700	28.2	4700	28.8
2000	26.1	2800	26.7				
1800	29.0	2600	28.7				

From the above values of H and d, plot the four curves W, W_1, W_2, and W_3 as shown in Fig. 17. These are the sag-tension curves.

Since the maximum tension will occur under the heaviest load and at the lowest temperature, the actual tension in this case should be limited to 5000 lb under loading W_3. A maximum tension of 5000 lb for this span and loading is equivalent to a horizontal tension of 4955 lb, which is the starting point for the pull-up curve at 0° F.

20. Pull-up Curves. The sag under a horizontal tension of 4955 lb and under loading W_8 is 27.38 ft. From Eq. 29 the length of the conductor under loading W_8, with a tension of 4955 lb and a sag of 27.38 ft is:

$$L = 800 + \frac{8 \times 27.38^2}{3 \times 800} = 802.499 \text{ ft}$$

Now assume a reduction in load which will reduce the tension by some convenient decrement, as 500 lb, and compute the change in length of cable and new length for a number of such decrements. By Eq. 31 the decrease in length per 500-lb reduction in tension is:

$$e = \frac{500 \times 802.499}{2{,}493{,}000} = 0.161 \text{ ft}$$

	Tension				
	4455	3955	3455	2955	2455
Original length	802.499	802.499	802.499	802.499	802.499
Reduction in length	0.161	0.322	0.483	0.644	0.805
New length	802.338	802.177	802.016	801.855	801.694

From these new lengths the corresponding sags are computed from Eq. 30 as follows:

For tension of 4455 lb, $d = \sqrt{\frac{3}{8} \times 800(802.338 - 800)} = 26.5$ ft

For tension of 3955 lb, $d = \sqrt{\frac{3}{8} \times 800(802.177 - 800)} = 25.6$ ft

For tension of 3455 lb, $d = \sqrt{\frac{3}{8} \times 800(802.016 - 800)} = 24.6$ ft

For tension of 2955 lb, $d = \sqrt{\frac{3}{8} \times 800(801.855 - 800)} = 23.6$ ft

For tension of 2455 lb, $d = \sqrt{\frac{3}{8} \times 800(801.694 - 800)} = 22.6$ ft

For these values of sag and corresponding tension the 0° F pull-up curve is drawn as shown in Fig. 17.

The next step is the determination of the changes in sag and tension as the temperature increases from its minimum of 0° F to its maximum of 120° F. This is done by plotting additional pull-up curves between these temperature limits. Additional curves have been drawn for 32°, 60°, 90°, and 120° F in Fig. 17 for this problem.

The starting point of the 32° F pull-up curve is the length of the cable at 0° F with a tension of 4955 lb, which was found to be 802.499 ft. With the loading unchanged, the increased length due to an increase in temperature from 0° F to 32° F is found from Eq. 31 as follows:

Original length	802.499 ft
Change in length, $e = 32 \times 802.499 \times 0.0000064 =$	0.241 ft
New length at 32° F	802.740 ft

The corresponding sag from Eq 30 is:

$$d = \sqrt{\tfrac{3}{8} \times 800(802\ 740 - 800)} = 28\ 7\ \text{ft}$$

This is the sag corresponding to a tension of 4955 lb at 32° F and is a point on the 32° F pull-up curve

By reducing the tension by equal decrements, computing the change in length by Eq. 31, and finding the new lengths and corresponding sags as was

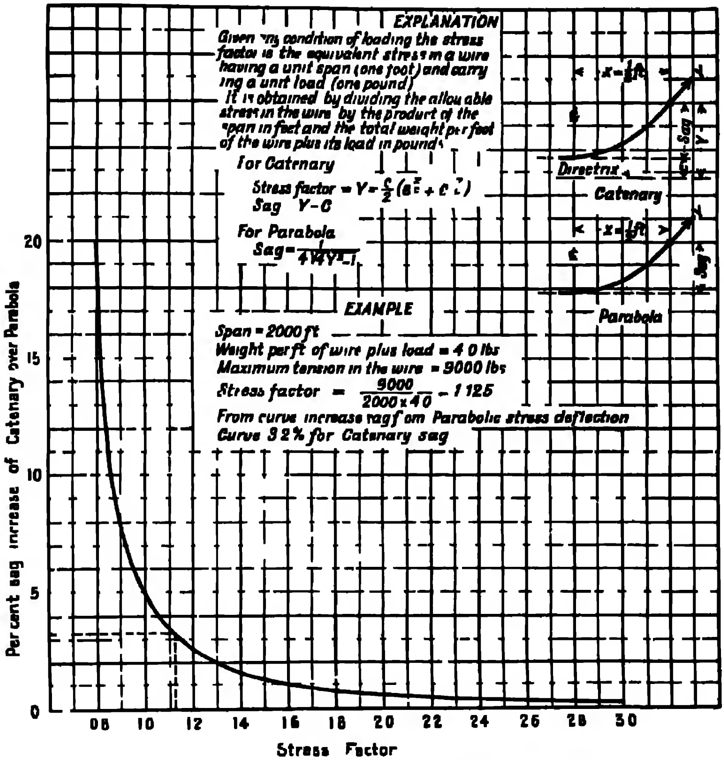


FIG 18 Comparison of parabola and catenary

done for the 0° F pull-up curve, additional points are obtained from which the 32° F pull-up curve is plotted

The remaining curves are plotted in a similar manner

Pull-up curves from 0° to 32° F inclusive must be so drawn as to intersect all sag-tension curves, but pull-up curves for temperatures higher than 32° F need not be drawn to intersect sag-tension curves for ice loading.

From the curves it is found that the maximum vertical sag is 27 1 ft and occurs at a temperature of 120° F and no wind, this sag being greater than the vertical components of the maximum sags under loadings including wind

pressure. The maximum side swing is 20 ft, and it occurs at a temperature of 120° F with a 15-lb wind on the bare cable.

The figures for maximum vertical sag and horizontal side swing for the most economical span should be used in determining the heights of towers and the normal width of right-of-way.

21. Catenary Solution. If it is desired to compute the sags on the basis of the elastic catenary, they may be figured as described for the parabola and corrected by the percentage increase obtained from the curve in Fig. 18. For further information on mechanical features of design, see "Transmission Line Design, Part I: Mechanical Features," by F. K. Kirsten; and Part I, Section B, "Mechanical Design of Spans with Supports at Unequal Elevation," by G. S. Smith, *University of Washington Engineering Experiment Station Bulletin* 17, February 1923, and *Bulletin* 29, August 1924, Seattle, Wash.

22. Stringing Curves. It is essential to give the field forces information which will enable them to string the cables in such a way that under any of the assumed loading conditions the maximum tension cannot exceed the allowable. Probably the best way to present this information is by means of stringing curves which show the relation between the sag and the span or the relation between the tension and the span at various temperatures.

Points for plotting these curves are obtained from the stress-deflection curves for various spans and temperatures. Typical stringing curves are shown in Fig. 19.

23. Structures. A decision as to the character of supports to be used should be governed by the general conditions outlined under Sections 2, 4, 14, 15, and 16. Wooden structures, either poles or H-frames, may be satisfactory where permanency is not essential. However, it is practically impossible to obtain as high a factor of safety with them as with steel structures, and, owing to the comparatively short spans which must be used because of the limitations in height, the number of insulators is greatly increased, thus proportionately increasing the probability of interruptions to service. For further discussion of wood-pole construction, see *Standard Handbook for Electrical Engineers*, McGraw-Hill Book Company.

The more important types of supports may be divided into three classes:

Steel pole.
Flexible steel frame.
Rigid wide-base steel tower.

Steel poles generally are used where space limitations will not permit the use of frames or towers and wood poles are not desirable. They usually are intended to take care of vertical loads combined with horizontal loads across or at right angles to the direction of the line with little or no provision for loads in the direction of the line. When designed for such longitudinal loading from broken or unbalanced wire pull, they must do the work of a tower; but because of the small dimensions at the base they require a larger amount of steel and heavier foundations.

Flexible steel frames, commonly called A frames, have been used where cheapness of construction was important. Like poles, their chief function is to carry vertical loads and loads across the line, and they depend on the cables to transmit longitudinal loads to heavier structures placed at regular

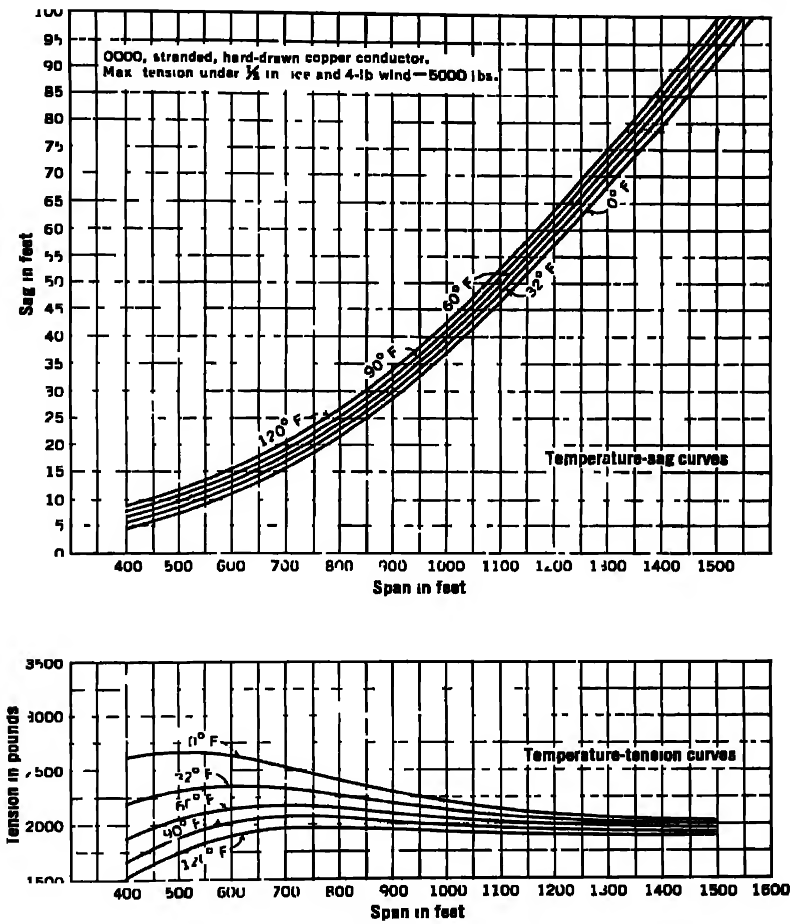


Fig 19 Stringing curves.

intervals along the line. Their use, in general, is not recommended, as their lack of resistance to load in the direction of the line may add to the difficulties of stringing the wires and, in cases of failure of one frame, all frames between it and the nearest strain tower generally are pulled down. If light structures must be used, a 4-leg tower of rectangular cross-section, weighing very little more than a flexible frame, can be designed to give the same strength across the line and considerable resistance to longitudinal loads.

Rigid towers provide the maximum security against interruptions to service caused by tower failure. Their strength allows the use of long spans with a consequent reduction in the number of insulators and in the probability of insulator failure. General practice has been to divide structures of this sort into the three following types:

Suspension line towers.
Suspension angle towers.
Dead-end angle towers.

Suspension line towers are used on tangents in the line and are designed to support unbalanced pull due to one or more broken conductors, in addition to loads from wind and ice on the cables.

Suspension angle towers located at angles in the line where suspended position of conductors is satisfactory are designed for the loads used for suspension line towers and the additional transverse loads due to angles in the line.

Dead-end angle towers are located at large angles in the line where suspended position of conductors is unsatisfactory and at other points where desirable to dead-end the line. They are designed to take the dead-end pull from any or all cables together with the loads due to wind and ice on the cables and the additional transverse loads due to angles in the line.

24. Tower Design. From the standpoint of design no definite line can be drawn between steel poles, flexible frames, and towers; the present discussion will be limited to the type of structures generally called towers, having trussed framework, the corner posts being supported on separate foundations.

The outline and dimensions of the tower depend largely on the arrangement and spacing of the conductors, the minimum clearances from the ground to the lowest conductor, and the separation of conductors and ground wires if ground wires are used. The separation of conductors and clearances from conductors to tower members and the position of ground wires and their separation from the conductors are determined almost entirely by electrical considerations and the degree of lightning protection which is intended. Clearance of cables from the ground is determined by the distance necessary for the protection from accidents to individuals and for uninterrupted service. It should not be less than that specified by the National Electrical Safety Code.

Where two 3-phase circuits are carried on one set of towers, the conductors usually are arranged vertically as shown in Fig. 20*a*. The conductors of one circuit are on one side of the tower with the upper and lower ones in the same vertical plane and the middle one offset outward from the tower. Where one circuit only is carried on a set of towers, common practice for high-voltage lines has been to place the conductors in a horizontal plane as shown in Fig. 20*b*.

The analysis of stresses and solution of problems involved in the details of design of transmission towers are not simple and should be intrusted only to an experienced designer familiar with the calculation of stresses in framed

structures. Great care should be taken to see that the combination of loads which produces a maximum stress is found for each tower member, as loads that produce the greatest stress in some members may not give maximum in others. Stresses should be computed for the following loads:

(a) Wind pressure on the tower applied at panel points of the tower.

(b) Pull from broken cables in the direction of the line, wind on the cables across the line, and vertical loads due to weight of cables and insulators plus ice coating, all applied at the points of attachment of cables.

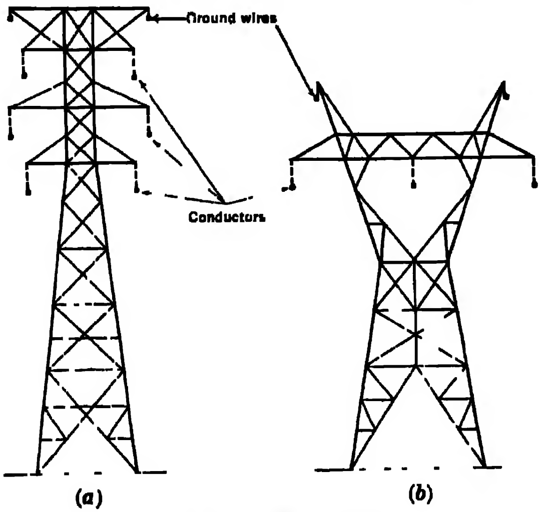


FIG. 20. Types of towers.

(c) Pull across the line from angle in the line, applied at points of attachment of cables.

(d) Weight of the tower itself.

Graphical methods of analysis are generally used as they are sufficiently accurate and less laborious than mathematical solutions. Stress diagrams are drawn, and the maximum combined stress in each member, for all loads, is found. The problem then consists in choosing appropriate structural sections to give the required strength, and in connecting them in such a way that the strength of the members will be maintained.

25. Unit Stresses. The unit stresses to be used in determining sizes of members in a transmission tower depend on the amount of overload that it is desired to have the towers capable of carrying. Standard-grade structural steel to be used in the fabrication of transmission towers should be manufactured by the open-hearth or electric-furnace process and should meet the requirements of the "Standard Specifications for Structural Steel for Bridges and Buildings," as given by the American Society for Testing Materials, A.S.T.M. Designation A7-42. See Table 4.

TABLE 4

YIELD POINTS OF STEEL, POUNDS PER SQUARE INCH *

	Standard Grade	High-elastic-limit Grade
Shapes:		
Tension	33,000	45,000
Compression	38,000, $160L/R$ For L/R less than 150	45,000, $190L/R$ For L/R less than 150
Tension	30,000	40,000
	30,000, $105L/R$ For L/R from 150 to 200	36,000, $130L/R$ For L/R from 150 to 200
Bolts:		
Shear	33,000	
Bearing	60,000	
Rivets:		
Shear	33,000	
Bearing	60,000	

* American Bridge Company.

Unit stresses under loadings required by the National Electrical Safety Code should not exceed those specified in that code for the given loading.

For further discussion of the subject of the design of transmission towers, including structures, loading, and unit stresses, see Ref. 7 and Chapter 40, Ref. 18.

26. Location of Towers. Suitable location of towers on the profile can best be determined by sliding a template of the maximum sag curve along the profile until the desired position of the span is found. The template should be made of transparent material, such as Celluloid, and should show, in addition to the sag curve, minimum clearance to ground and lengths of spans all drawn to the same scale as the profile.

Figure 21 shows curves for making such a template and a section of profile illustrating its use. It is well to show a minimum sag curve on the template and to locate towers so that there will be no uplift on them under minimum sag conditions. When uplift cannot be avoided, the towers must be made to resist it.

It often happens that towers somewhat higher than those used for normal spans are desirable at certain locations to give the most economical arrangement. To meet these requirements, tower extensions are provided so that the towers can be made of a standard height.

27. Protective Coating. It is practically impossible to paint the portions of a tower above the level of the lowest conductors without de-energizing the line, and it has generally been found more satisfactory to use a galvanized structure. This has led to the use of bolted towers, as satisfactory galvanizing cannot be done on riveted work and riveting after the galvanizing has been done destroys the galvanizing near the rivets.

For special high towers with very heavy loading, where bolted joints would not be satisfactory, it is possible to rivet and paint the part of the towers below the conductors and galvanize the portion above.

Specifications for galvanizing should be rigid, and all galvanizing should be given very careful inspection.

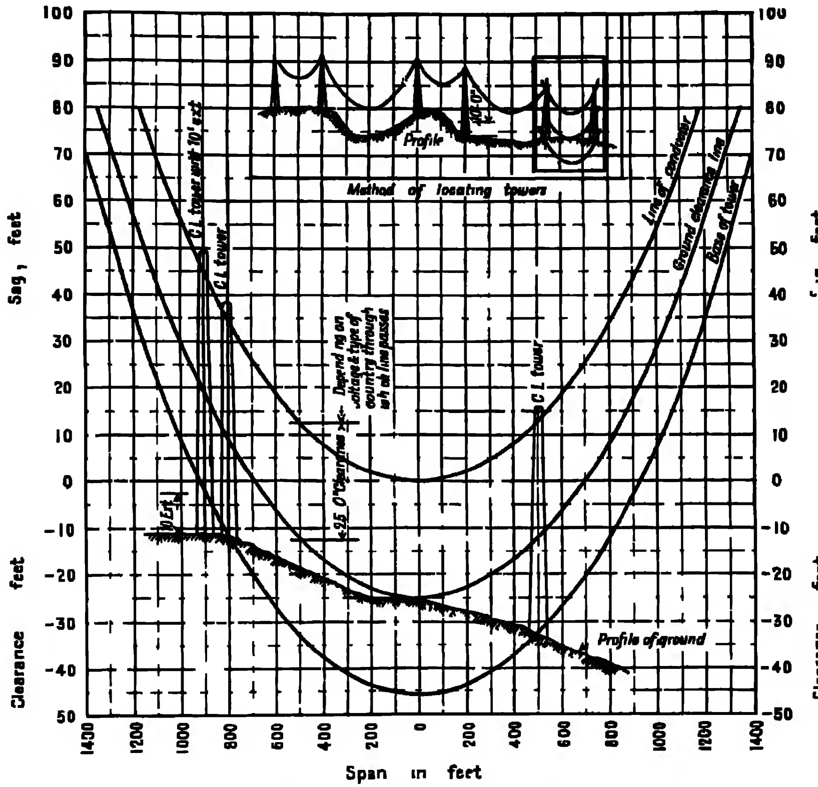


Fig. 21. Tower location diagram.

28. Tower Foundations. In general, there are two types of tower foundations, concrete anchors and steel anchors. The individual anchors for each leg of a tower should be designed to resist the maximum uplift or compression in the leg plus that from the possible overload.

A steel anchor consists of a structural member or members extending into the ground and connected to a steel grillage. The grillage should be of sufficient size to transmit the maximum compression to the soil without exceeding allowable bearing pressure, and it should be buried to such a depth that it will resist the uplift from the tower leg plus any desired overload. In general, the resistance to uplift may be assumed equal to the weight of earth

of an inverted frustum of a pyramid with a bottom area equal to the area of the grillage, its sides forming angles of 30 degrees with a vertical plane and its height equal to the depth from the surface of the ground to the grillage.

Concrete anchors should be designed to engage the surrounding earth so that the earth will resist uplift in combination with the weight of the concrete.

29. Outdoor Station Structures. Another type of structure, in the same general class with steel transmission towers, is the steel structure used for supporting busses and switches for outdoor substations and switching stations.

The general arrangement and dimensions of these structures are usually determined largely by electrical considerations. The loads which they must support are those produced by the incoming or outgoing lines, the pulls from suspended busses, dead loads from switches, insulators, and structures, and wind on structure and equipment. Sleet formation should be given the same consideration as for the transmission line.

Some of the less important low-voltage structures are painted, but, because of the inconvenience of repainting, galvanized steel is generally used for high-voltage structures (see Chapter 42, Section 15).

30. Bibliography. (See also Chapter 40, Section 17, for general bibliography.)

1. ERNEST LEE PANNELL, *High Tension Line Practice: Materials and Methods*, D. Van Nostrand.

2. L. E. IMLAY. *Mechanical Characteristics of Transmission Lines*, Westinghouse Technical Night School Press.

3. L. F. WOODRUFF, *Principles of Electric Power Transmission*, John Wiley & Sons.

4. WILLIAM NESBIT, *Electrical Characteristics of Transmission Circuits*, Westinghouse Technical Night School Press.

5. *Handbook for Overhead Line Construction*. National Electric Light Association, Franklin Printing Company, Philadelphia.

6. L. V. BEWLEY, "Protection of Transmission Lines against Lightning: Theory and Calculations," *Gen. Elec. Rev.*, April and May 1937.

7. *Transmission Towers*, handbook, American Bridge Company.

CHAPTER 44

OPERATION OF HYDROELECTRIC PLANTS

*By Albion Davis**

ORGANIZATION

1. General Plan. The successful operation of a hydroelectric plant is largely dependent on the organization plan set up originally for the purpose. This plan will include not only the organization itself but also the principal tools with which it will work, namely, instruments, communication facilities, equipment test data, current data for forecasting, records, and reports. The initial plan is important because, once established, procedure tends to crystallize, and changes become more difficult.

2. Organization and Personnel. The first step is the selection and training of the personnel that will actually operate the plant and maintain continuity of service. In developing the organization plan it should be kept clearly in mind that the load-dispatching or system-operating group is the heart of a power system and that every other part of the plant organization must be tuned to function successfully with it.

The size of the organization depends upon the capacity of the plant, the number of units, and the complexity of the station layout, as well as upon whether the plant will be automatically operated or whether it will be entirely a manually operated plant. The size of the organization also depends on the extent to which the plant will be called upon to handle its own maintenance, its own accounting, purchasing, real estate, or legal work.

For a specific plant, the organization will naturally be adjusted to the system plan and the general procedure of the company which will control the project. Old plants usually require a larger personnel than modern plants.

In a plant of moderate size there will generally be four divisions of the operating force: (1) The operators, concerned with operation of plant generating equipment, control of power fed from the plant, and operation of river control equipment. These men usually work in shifts so that an operating force is on duty at all times. (2) The electrical maintenance men. (3) The mechanical maintenance men. (4) A plant engineer concerned with forecasting, plant efficiency, and load control plans.

Figure 1 shows a plant organization chart that might be suited to a modern 160,000-kw low-head hydro plant having not over eight main generating

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units and these of the same size and type. It would be for a plant of simple design feeding a limited number of outgoing circuits; a plant where all equipment to be operated is located within or adjacent to the plant; a plant that is to handle its own maintenance and minor construction but not the real estate, purchasing, or accounting work usually handled by a central office.

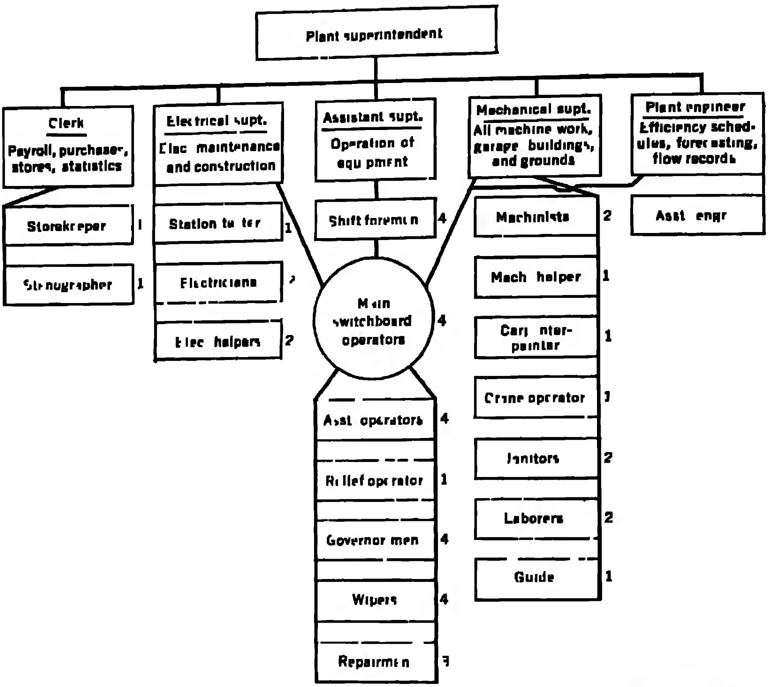

FIG. 1. Plant organization chart suited to a modern eight-unit 160 000-kw. low-head hydroelectric plant of simple design. It excludes transmission substation accounting, real estate, and general office requirements.

3. Plant Operators and Maintenance Men. Since the size of the operating crews will be determined by requirements in emergencies, plant equipment should be designed to give dependable service. Simplicity and ruggedness of parts are essential, even in the details of design, if emergency requirements are to be reduced to the minimum. Furthermore, the design should centralize control as much as possible and provide conveniences for rapid inspection and repairs. Location of equipment, number of operating floors, arrangement of the trash racks, provisions for cleaning them and for protecting against ice—all have a bearing on the number of operators or maintenance men needed in emergencies. To attain low operating costs, close cooperation between the designer and the operating department is advisable while the plant is under design.

The modern tendency is to operate small stations by means of semi-automatic equipment and supervisory control. Such stations require only periodic visits by an operator, who may handle two or three small stations. Sometimes plants as large as 20,000 kw are operated by remote control in this manner. For manually operated plants up to 20,000 kw, one operator per shift or 4½ operators per plant are ample for all ordinary conditions if they do not have to take care of heavy maintenance work. Where there are a number of plants, many systems effect economies by utilizing maintenance crews that move from plant to plant.

In compactly arranged plants of 100,000-kw capacity from 4 to 6 operators per shift will usually be required. For a plant of 200,000-kw capacity, from 6 to 9 operators per shift may be required. The total number of operators for plants of this size may vary from 18 to 36. Such plants will usually have from 3 to 5 electrical maintenance men and 4 to 6 mechanical maintenance men. The total force of operators and maintenance men, including superintendence and miscellaneous labor, need not exceed from 30 to 55 men for plants of this size.

Operators will be chosen particularly for their experience with power-plant machinery, their alertness, and their ability to think straight in emergencies. For a new plant the main switchboard operators will usually be thoroughly experienced, capable men drawn from other plants. Some of their assistants may be men who have worked on the electric installation during construction. A few may be younger men from the local community who later will work up through the ranks. Though it is not a necessity, it is helpful if the operators have had some technical training in trade schools or colleges.

Proper training of the operating force both for normal operation and for emergencies is, of course, vital to successful operation. Every operator selected should be required to prove by examination his understanding of the equipment he is to operate. Provision should be made for thorough schooling of the electrical operators, particularly in the handling of hydraulic equipment. As a part of their training it is very desirable that the key men of the operating organization work with the construction test crews for a time before operation begins. It is particularly helpful if one or more of the maintenance men has been on the job during the construction of the plant.

4. Communication Facilities. For successful operation adequate communication facilities are necessary not only to maintain service in emergencies but also to permit the easy flow of information between plants and the central office at all times. The communication facilities may include interplant telephones for the larger stations, private or leased telephone lines between plants, carrier-current circuits, and in some cases radio. There is probably no other single factor that aids more in securing smooth and efficient operation than good communication channels between all the operators and engineers who are connected with daily operation.

If communication facilities are limited, a plant may still operate satisfactorily but nevertheless at poor efficiency in relation to the system as a whole.

Lack of ready communication has all too often relegated plants to the mere grinding out of kilowatt-hours to use up the water available, regardless of how much those kilowatt-hours earn in the system load curve. With adequate communication facilities, or their equivalent in automatic operation, an otherwise isolated plant becomes an integral part of the system and just as important as any other generating plant. From the system operator's standpoint, the most important plant is the one which he can get on and off the line with the least effort. Good facilities for this purpose should certainly be considered a vital part of any project.

5. Engineering in Operation. It is highly desirable, in the operation of hydroelectric plants, to have an engineering control which parallels the operating organization from top to bottom. This control should be permitted to cut across department boundaries and effect a coordinated plan of system operation. The engineering control would be the planning agency; the operating organization would be the doing agency of the system. Each within its sphere should have latitude and authority. The one would be responsible for efficient plant operating schedules and the most efficient use of hydro plants as system units; the other would be concerned, ahead of everything else, with maintenance of service and upkeep of equipment to attain that end. In a small plant, all functions must be combined in one man having the required qualifications. The exact form which the engineering control takes depends on the size of the system of which the plant is a part, as well as on the type of organization at the central office.

For a 100,000-kw plant with a wide range of output, a considerable sum should be spent for forecasting alone. In such a plant, certainly one, and perhaps two, engineers may be justified by the economies they can effect. For a large plant with storage, an expenditure of \$3000 to \$4000 annually for maintenance and operation of rainfall and river gages may be justified for forecast and record purposes, whereas, in a small plant, such an expenditure would be completely out of line.

The engineering personnel should be kept in balance with the amount and quality of the engineering data to be secured, the instruments, tests, forecast data, and records that are to be provided. It would be as useless to have engineers without engineering data as it would be to have elaborate engineering data without the personnel to interpret them. As aids in analyzing system performance, the larger systems have made increasing use of graphic instruments, such as those for recording voltage, frequency, or kilowatts at high chart speeds during disturbances. In many of the larger plants such instruments as automatic oscillographs, meters for totalizing plant output or integrating that of several plants by telemetering equipment, and wide-scale frequency recorders have proved desirable.

6. Plant Equipment Ratings. In planning for operation, another essential is to provide for the development of adequate water-wheel and generator ratings. The water-wheel tests should cover all gate openings and should be

made at all speed-head ratios that will occur in operation. Otherwise, when the head differs from the test head, it will not be possible to use the turbine accurately as a water meter. The water-wheel tests can best be made in connection with the acceptance tests of the generator and water wheels. The generator tests can readily be made to cover all possible loadings. Money spent initially for adequate tests will be repaid during a short period of operation.

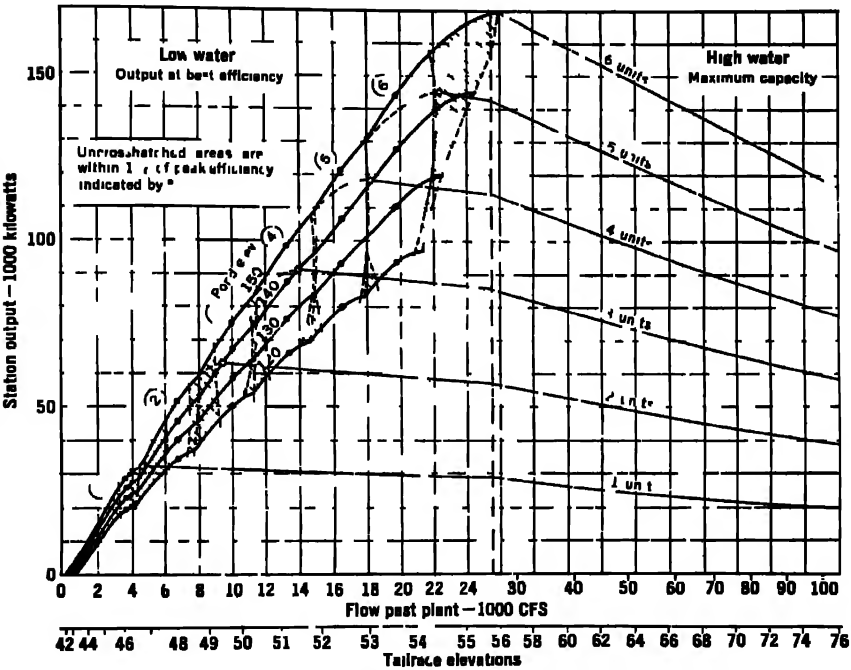

Fig. 2. Plant rating curves for a six-unit 150 000-kw project that would be subject to capacity reductions in high water due to reduced head.

After complete water-wheel and generator tests, it then becomes possible to build up any number of useful plant-rating curves. An example of a plant-rating curve is shown in Fig. 2. Such curves usually will be developed the first year or two of operation as the need for one after another of them becomes apparent. With them, a complete accounting of water used through the plant is possible. Without them, the plant operating force is considerably handicapped in obtaining maximum economy.

At the time the water wheels and generators are tested, the governors should also be tested for performance under all the conditions they will meet in operation. These tests should include not only the usual tests for load-on

and load-off speed changes but also tests for sensitivity and accuracy of response to speed changes during normal operation.

In many plants there will be other equipment that may be expected to limit the efficient output of the plant. Such equipment should be tested and rated soon after its installation. Under this category might come tests of the heating limitations of transformers and tests of transmission-line stability.

The engineering organization responsible for the design and construction of the project should gather and turn over to the operating organization complete detail plans of the plant as actually constructed, together with manuals covering all machinery and equipment with the manufacturer's recommendations as to care and maintenance.

7. Hydrographic Data. Another factor of prime importance in the organization plan is providing for an adequate system of forecasting river flow and power available. This will require the establishment of rain gages in the watershed above the dam and a number of river gaging stations. The number and location of these rainfall and river gages will, of course, depend on the size of the watershed and the type of plant. The needs of a run-of-river plant will be entirely different from those of a plant with considerable storage. In establishing gages, close cooperation with state and federal agencies is very desirable.

Additional river gages will be needed to record backwater levels in the reservoir area for use in connection with possible damage claims. It is wise to have these gages installed a year or two before the raising of the reservoir level so that direct comparison of water levels before and after the dam can be made. Also it is well to make a survey of high-water marks before the construction of the dam because many of them may be lost by the raising of water levels, by death of the observer, or by his moving away. In some cases high-water-mark testimony has been perpetuated by the deposition of witnesses in court proceedings during the early period of operation.

Along this same line, it is a good plan to make provision for securing cross-sections of the reservoir at critical points—particularly at bridge locations over the reservoir, where the cross-sections may be reproduced later for a record of silting. These cross sections should be taken in a manner that will permit their reproduction from permanent marks on the ground, even after a lapse of many years.

Provision should be made for the installation of automatic gages for water levels about the plant, more particularly, the headwater level and the tail-water level in locations least influenced by the velocity of flow to or from the generating units of the plant. Where the pool is large and the computation of inflow involves estimating daily pondage, it is often advisable to install a wind-velocity and -direction recorder so that the natural pool level without wind may be determined with a fair degree of accuracy. In very large reservoirs, evaporation may be a factor of importance, in which case the installation of a standard evaporation pan may be desirable to furnish the means of making a complete accounting of the water storage.

8. Operator's Records and Reports. Provision should be made for taking readings of indicating instruments at least every hour, preferably every half hour, throughout the 24, and for one log sheet for every 24 hours to be sent to the main office for study and filing. Every case of unusual trouble should be reported minutely with all related facts. Any failure of apparatus should be fully covered with a report, and a copy of this report should be noted on the apparatus card in the designing engineer's office. A study of these cards at any time will give a complete history of each piece of apparatus. Such studies are particularly useful as collective information for groups of similar apparatus or for a comparison of the apparatus made by different manufacturers.

Log sheets are designed to suit the requirements of the particular type of plant. Samples are easily obtained from any of the operating companies. In addition to the regular readings of generator, transformer, and circuit watt-hour meters at least once a day, the log sheet will usually include the following readings at half-hour intervals:

1. *Generator* amperes, kilowatts, reactive volt amperes, exciter amperes, exciter volts, and temperature.
2. *Turbine*-gate openings and bearing temperatures.
3. *Transformer* amperes and temperatures.
4. Station kilowatts, voltage, frequency, headwater level, tailwater level, spill-gate openings, lines in service, and weather conditions.

The importance of a station may justify plotting curves of wind direction and velocity, records of hourly precipitation, head- and tailwater levels, river flow, load, and capacity on the load—all of these preferably on a weekly curve sheet. The weekly chart provides a very useful record of plant operation. When filed, all the Sundays and other days of the week stack up one above the other and all the characteristic variations by days show up nicely in their relation to each other.

In order to picture properly the part each plant plays in the system economy, the central office will need some of the data from the plant log sheets for plotting composite system load curves and other details of operation. The data secured by the central office daily from each plant will include total kilowatt-hours, water used through the plant, water wasted over the spillways, total water used, average tailwater readings as a check on total discharge past the plant, headwater readings for use in determining storage, and such other information as will give the central office an accounting of the water used by the plant.

In addition to the log sheets, the operators should make periodic inspections and reports on the general condition of the plant. This is particularly advisable where settlement, leakage, sloughing of banks, scour, washouts, and other sources of trouble may develop. These inspection reports by the operators would be largely for calling immediate attention to impending difficulties that may require more detailed study by the central office. These periodic reports

by the operators should be sufficiently detailed to insure that the operators are vigilant. Daily inspection of certain items is necessary; for other items weekly or perhaps monthly inspections may be sufficient.

9. Cost of Operation. Finally, the personnel and control facilities to be provided in any organization plan should be scrutinized carefully as to cost. The number of operators, the size of the maintenance crews, the cost of engineering, communication facilities, forecasting, and particularly the cost of routine records for the central office should be justified by what each contributes to system earnings. Usually it is wise to start the organization plan on the small side and build up later with new material that clearly justifies itself, rather than to start out big and have to remove dead wood later. Although costs of operation and maintenance will vary considerably according to the location and type of plant, it is of value to have some guide against which a proposed organization plan can be measured.

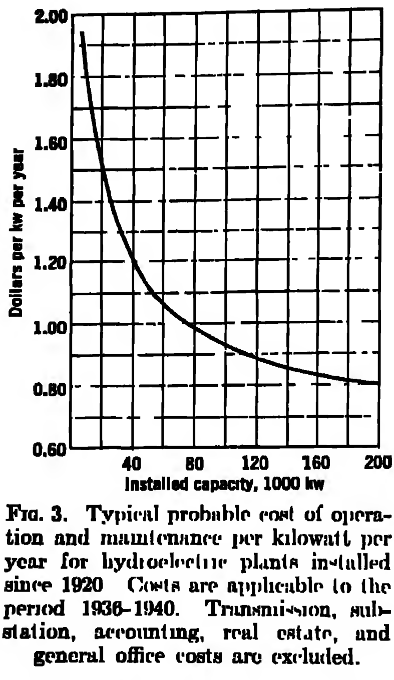

FIG. 3. Typical probable cost of operation and maintenance per kilowatt per year for hydroelectric plants installed since 1920 Costs are applicable to the period 1936–1940. Transmission, substation, accounting, real estate, and general office costs are excluded.

Annual costs for operation and maintenance are more or less proportional to the capacity of the plant and the number of units. There are some plants of 5000- to 10,000-kw capacity where annual operating costs before World War II ran from \$4 to \$7 per kw of installed capacity, yet there are other plants of the same size where annual costs were less than \$2 per kw. A modern plant even as small as 5000 kw, if it has a reasonable amount of automatic or remote-control equipment, need not cost more than \$12,000 to \$15,000 annually for operation and maintenance. For large modern plants of 100,000- to 200,000-kw capacity, the annual cost of operation and maintenance may be as low as \$0.75 to \$0.90 per kw of capacity. In older plants with a larger number of units and different types of units, annual costs run considerably higher.

In *Power Supply Economics* the authors, Justin and Mervine, give a composite picture of plant operating costs that is representative of some thirty plants in different sections of the country. The tabulation is a revision of their figures, the increased labor costs of more recent years being considered. This revised table may be taken as typical of annual costs for the period 1936–1940 for operation and maintenance of plants installed since about 1920. Figure 3 shows the same data in graphic form.

Capacity of Plant, kw	Typical Probable Total Annual Cost of Operation and Maintenance at the Plant	Typical Probable Cost of Operation and Maintenance per Kilowatt per Year
10,000	$ 18,000	$1.80
20,000	30,000	1.50
40,000	48,000	1.20
75,000	75,000	1.00
125,000	110,000	0.88
200,000	160,000	0.80

These figures exclude transmission and substation costs, also those related to accounting, purchasing, real estate, and other general office costs chargeable to the project. The costs given above should be used with caution and only for a preliminary checkup of the organization plan.

Operation of Equipment

10. Duties of Plant Operators. There is a wide distinction between system operation in the sense of maintaining service to customers and system operation in the sense of securing the lowest possible cost of power to the system. The first type of operation will be distinguished by the heading "operation of equipment"; the second, by the heading "hydraulic operation." These two types of operation will be treated separately.

Under the general orders of the load dispatcher, or system operator, the plant operators control all generating equipment and all apparatus regulating the flow of electric power to or from their plant. They watch lubrication, bearing temperatures, generator and transformer temperatures, and adequacy of cooling water supply, and they see that none of the equipment is operated beyond safe limits. They report operating conditions to the load dispatcher at assigned regular intervals. The operators raise and lower crest gates and control river levels under the general direction of the plant engineer or the person responsible for hydraulic operation.

The operators should be supplied with detailed emergency instructions, including those for short circuit, fire, floods, and injuries to operators, especially electric shock. Their instructions should cover the immediate steps to be taken in restoring service regardless of whether communication is available or not. These instructions should cover all types of interruptions which can be foreseen.

The operator in charge of a shift must not only have a solid background of operating experience and a detailed knowledge of all the hydraulic and electrical equipment in his own plant but he must also have a knowledge of the operating facilities of the system to which his plant is connected. He must know the limitations of all generating, transmission, and substation equipment. Above all the operator must be able to meet the test of clear thinking in sudden emergencies. Training of the operators should center around how

to prevent outages and how to restore service promptly if outages occur. This training should first include thorough grounding in the handling of hydraulic equipment, particularly pumps and governors, because they must be operating properly before service can be restored.

Each operator should be given a complete description of the development, and a set of plans, as well as instructions for operating special pieces of apparatus, sluice gates, head gates, crest gates, and similar equipment so that, no matter what his regular duties may be, he may know the duties of others and prepare himself for advancement and for emergencies that may arise. In this connection an instruction department for a large power system is well worth while. The duties consist of furnishing each operator with complete and accurate information about the particular plant that he operates, and then making sure that he understands his instructions perfectly and memorizes them. Unexpected examinations at the plant provide the follow-up that is necessary. The instruction department may provide further incentive for improvement by conducting bogey contests for generating efficiency with prize awards periodically to the winning groups.

To help in keeping the men up to date as well as to promote a spirit of cooperation it is distinctly advantageous to arrange for operators to visit other plants on the system and the men with whom they deal over the communication lines. Every plant should see that technical magazines are made available to the men. Some companies go so far as to provide for regular classes in mathematics, physics, electricity, and hydraulics.

11. Dispatching of Loads. All instructions for the operation of the generating and transmission system center in the load dispatcher. His primary duty is to maintain continuity of service from the generating plants to the substations, and he will give general orders to that end to the plant and substation operators. He will issue permits and releases on system equipment. He will give specific orders for restoration of service in emergencies. He will designate the loads each plant is to carry, the amount of reserve to be provided, and the extent to which it is to carry the speed regulation of the system. In assigning loads he will be guided by general plans for system economy, yet he will leave as much latitude as possible for the plant organizations to work out their best economies.

The actual dispatching of loads is best done orally, by telephone, or wired wireless, by one man only. He must have had extensive actual experience in operating. Nothing less will do. On a large system, in case of emergency, part of the dispatching may be delegated by the chief dispatcher to division dispatchers located closer to the plants and consequently having better communication with the plant operators.

A load dispatcher should be on duty every minute of the year, usually working in shifts of 8 hours each. His main equipment consists of direct telephone or wired-wireless communication with every plant or substation operator on the system, a miniature diagram of the entire power system, and instruments recording frequency and cumulative time error. Radio equipment is gen-

erally provided for receiving weather reports and time signals. These facilities may be supplemented on large systems with some form of telemetering or long-distance recording in the load dispatcher's office of such strategic data as tie-line loads, main generating-station loads, voltage at controlling points, and river levels.

The miniature system diagram in the load dispatcher's office will show every generating unit, every switch between the generating units and the low-tension side of all substations, and the connecting transmission lines. All apparatus that is alive in the system shows a red light or a red plug on the board. These are set by the load dispatcher in accordance with his orders to the operators. This gives a visual representation of the entire system for the use of the dispatcher and also apprizes the relief dispatcher of conditions when he starts his shift.

The load dispatcher's office is the clearinghouse through which all plans for maintenance of major equipment pass. Generally the load dispatcher issues permits or releases of main generating units, transmission lines, and substations, but the actual issuance of hold-off cards for tagging equipment not to be operated is usually handled by the main generating plant operators or by division load dispatchers—each for his own station or section of the system. For the protection of the men working on the lines or equipment, the system of providing clearances must be covered by rigid and inflexible rules of procedure.

The load dispatcher will receive hourly or half-hourly reports from all operators on the system in order that he may know fully all the generating, transmission, and weather conditions. He will endeavor to anticipate all unusual operating conditions and be prepared to meet them. Dark clouds over a city, a bad storm, a sudden flood, or sleet may require sharp changes from the schedules he previously assigned for normal conditions. The load dispatcher is usually able to predict storm conditions accurately by knowing the time, place, direction, and velocity of the storm when it first strikes his system.

12. Emergency Operation. To protect service during storm periods, it has been common practice to cut a large system into several sections operating independently, so that the effect of lightning flashovers may be minimized. The loss of efficiency for the period of the storm has usually not been a major consideration. Within recent years many systems have been provided with adequate relay protection and high-speed switching and now operate tied together throughout storm periods. Many of these systems, however, continue to carry a sufficient amount of rolling reserve so that, in the event of a break, each section can carry its own load. Companies which have equipped their section tie lines with overhead ground wires and underground counterpoises, expulsion gaps, etc., consider the lines practically lightningproof and avoid operating the extra section reserve.

In addition to having fixed instructions for restoring service in his own station, each plant operator should have rigid instructions for energizing lines to adjacent stations in emergencies, without waiting for authority from the

central load dispatcher. For their own protection, all plant and substation operators and all patrolmen on the system should memorize these instructions.

The broad problem of system operation in emergencies is outside the scope of this chapter. It is a study in itself. For such an analysis reference is made to an excellent article by William R. Hamilton, entitled "Planning for Emergency Operation," in the *Edison Electric Institute Bulletin* for December 1936.

13. Plant-loading Schedules. It is desirable that the load dispatcher prepare his plant-loading schedules on the day preceding their application. The load dispatcher's estimate of the load to be carried will take into account the day of the week, the weather conditions expected, whether the temperature will be higher or lower than normal, and probable changes in major industrial plant operation. In his assignment of loads to the individual plants the load dispatcher on a large system may be guided by a general load-allocation plan supplied by the system planning head. Factors entering into such a general load-allocation plan are discussed under "System Use of Hydro" in Sections 44 to 55, inclusive.

The load dispatcher will first take into account river conditions at the hydro plants, to be sure that all energy available from them will be used, and then assign the balance of the load to steam plants. In his plant-loading schedules, the load dispatcher keeps in mind schedules for planned outages for maintenance. Steam-plant outage schedules for maintenance of boilers and condensers are an important factor.

It is customary for the load dispatcher to have his plant-loading schedules for the following day made available to the plants by three o'clock in the afternoon. This gives each plant crew adequate time to plan their maintenance work on idle units and to see that the necessary supplies and personnel are available. The actual load they may be called upon to carry the next day may vary somewhat from the schedule, but, if there are several generating plants on the system which can divide the change in load, the difference between scheduled and actual load will be small.

14. Reserve. The matter of providing adequate reserve to cover possible emergencies will govern to a large extent the assignment of steam-plant loadings. It is common practice to provide a running reserve equal to the load carried by the largest unit of equipment which might fail. It is desirable that this running reserve be divided so that a proper share of it is available to each section of the system.

15. Control of Frequency. With the almost universal use of electric clocks the load dispatcher must not only maintain constant frequency but also keep accurate time from hour to hour. While hydrogovernors are now being built which, on isolated load, will automatically maintain close frequency and time, these governors may have to operate in parallel with less sensitive governors. Under such conditions, system frequency may oscillate above and below average several times a minute. Then, too, the average frequency may vary considerably with the load being carried, unless the governor settings are

periodically adjusted. This can be done either manually by the operators or by automatic equipment designed to do the same thing.

The load dispatcher will designate for the different periods of the day which plant is to be responsible for keeping accurate system time. Other plants will then regulate the load on their tie lines to the frequency-regulating plant. When a plant is assigned the responsibility of keeping accurate time, the operators may have to adjust the settings of the governors every 3 to 5 minutes. Where the load curve is relatively flat, the governors may carry accurate time for periods of 30 minutes or longer without time departing more than 1 or 2 seconds from standard.

Interconnections between large systems with relatively light tie lines have emphasized the importance of maintaining exact frequency on each section of the system so that the wider swings in tie-line loadings may be avoided. To do this, close analysis and coordination of the action of individual governors is necessary. It is to be expected that the load dispatcher will in the future be required to add to his duties the assignment of the individual settings of the governors on all major generating units. These settings will be determined by the contribution desired of the individual governor, first in stabilizing normal system speed; second, in bringing on rolling reserve quickly to replace loss of generating capacity; and third, in regulating load changes on the unit so that its incremental efficiency will be kept in balance with other units on the system.

16. Lubrication and Care of Hydraulic Turbines. For long life and efficient service, proper lubrication of every wearing surface of the turbine is essential. Yet the problem is not simple. It requires close attention to details on the part of the designer and intelligent care on the part of the operating organization to be sure the right grease is used and that it gets to each bearing regularly and in sufficient quantity. Turbines may run for years without attention, sometimes even without proper lubrication. Although wear is a rather slow process at first, and seldom involves an outage of a unit while needed for load, the over-all efficiency of the turbine may be very seriously affected in a relatively short time and the ultimate life and value of the unit seriously curtailed. In the design of the turbine, means should be provided for conveying grease to every important bearing surface from a convenient central location. Individual leads to each bearing are desirable when the lines are long or where the division of grease between bearings cannot be observed. High-pressure grease-gun systems such as the Alemite have proved effective in getting the right kind of grease through long leads.

The turbine-gate rigging contains many heavy parts in continual motion. There may be 20 guide vanes and as many sets of pins and links tying them to the shifting ring—altogether perhaps 100 bearings to watch. Much of the maintenance cost of a turbine centers around the proper care and adjustment of this mechanism. The pins and links on the shifting ring, where not submerged, do not require as much attention. The upper and lower

guide-vane bearings are the important places to watch. It is essential not only that these guide-vane bearings be adequately lubricated but also that the packing in the stuffing boxes on the guide-vane stems be kept in good condition to avoid leakage that might carry silt and other material into the bushings and induce very rapid wear. Cutting down the leakage will mean less silt and less wear.

It is a good plan to establish at the outset a lubrication schedule which will show the frequency of the lubrication and the quantity and grade of lubricant to be used. The amount of lubrication required will depend principally upon the amount of regulating which the turbine does. Whereas greasetight bearings can ordinarily be kept well lubricated by the use of grease guns once a day, turbines doing considerable regulating may require applications of grease three or four times a day even when the turbine is new and all fits are close. At the start a liberal but definite schedule of greasing should be followed at least until turbine inspections demonstrate that a decreased schedule would be adequate. Where the water used by the turbine carries considerable silt and corrosive material, more frequent applications of grease should be made. As wear progresses the greasing should be done at more frequent intervals. Most companies consider it wise to encourage the use of plenty of grease.

The grease should have a consistency that will assure uniform movement through the leads under all temperature conditions. Its composition should be such that it will not separate, cake, or harden in the leads or bearings. It should have no tendency to corrode. Any grease that is used in locations exposed to flowing water should have adhesive qualities and should show no tendency to emulsify when submerged for long periods. Greases containing fillers such as asbestos, mica, talcum, or chalk should be avoided because they will separate out and clog the passages. Sodium-base greases with their high melting point, good stability, and fiber-like structure are suited to places where the duty is heavy and temperatures may run fairly high, but they are not resistant to water. They should not be used for submerged bearings. Calcium-base greases that are resistant to water can be made, but care must be exercised in making a selection because many of them disintegrate gradually when submerged. Stable lead- or aluminum-base greases that have stocky, adhesive qualities and no tendency to slump or flow except under pressure are very satisfactory, particularly for the guide-vane bearings and for underwater service, because they are not affected by water. The fact that the aluminum-base greases have a tendency to become more cohesive with increase of temperature is an advantage rather than a disadvantage with the slow-moving turbine parts.

17. Care of Main Turbine Bearings. The thrust bearing requires close attention. It is the most heavily loaded and most important bearing on any generating unit. Means of measuring thrust-bearing temperatures should be provided as well as indicators for showing the flow of oil and of cooling water to the bearing. Readings of thrust-bearing temperatures preferably should

be taken from thermometers with their bulbs located in the bearing rather than in the oil. The readings should be regularly recorded on the station log sheets by the operators, and checks should be made by them at the same time for flow of oil and water. It is essential that some form of alarm be provided to give warning in case of failure of the supply of cooling water or oil. The alarms should be checked periodically to make sure they are always in working order. On heavily loaded bearings it is particularly important to watch the action of the bearing when the unit is started from rest, as this is the time when the bearing can most easily be damaged.

For turbine steady bearings of the water-lubricated lignum vitae or rubber type, sufficient water should be used to keep the entire bearing shell saturated. To be sure this is done it is desirable that some excess water be allowed to run out at the top. The amount of water flowing out of the bearing shell at the bottom through the packing gland or sealing device must be watched and not allowed to become excessive. In some cases the suction down through the bearing may be sufficient to draw the water right on through the bearing without letting it remain there long enough to accomplish saturation and proper lubrication. Renewal of the packing gland or sealing device should be undertaken before there is any danger of lack of sufficient water for lubrication. Where the water for the bearing is conveyed through pipes it will usually prove desirable to pass the water through a screen filter located where it can be cleaned conveniently. Operating convenience usually justifies a double screen filter so that one screen can be removed from service for cleaning without interrupting the flow of water. The screens should be cleaned at sufficiently frequent intervals to prevent clogging. The danger of frazil or cake ice clogging the screens should be guarded against particularly. Suitable devices indicating the flow of water should be provided and observed at regular intervals because lack of sufficient water will soon cause the bearing to burn out.

Where oil-lubricated, babbitt-lined steady bearings are provided, continuous circulation of the oil through the bearing is necessary. If the feed is by gravity from a tank above the bearing, the oil level must be continuously maintained. Lacking oil, the bearing will be ruined in a few minutes. Duplicate pumps for circulating oil are desirable. They should be arranged so that, if one pump fails, the second immediately cuts in, at the same time sounding an alarm, thus not only keeping the flow of oil continuous but also notifying the operator immediately of the trouble. Many of these bearings have been in use over a long period of years with very little operating trouble. One company reports having such bearings on two units accidentally submerged and kept running under load for several hours with only water for lubrication without damage resulting. This type of bearing should be provided with an oil-flow indicator, a low-oil-flow alarm, a bearing temperature indicator, and a means of observing presence of water in the bearing oil reservoir. Under certain river conditions there may be a tendency for water to enter the oil reservoir through the packing gland at the bottom of the bearing. To avoid

this, the packing gland should be kept in good condition and reasonably tight.

Readings of bearing temperature should be recorded by the operators at regular intervals, and at the same time the flow of oil to the bearing should be checked. If water coils are used for cooling the oil, the flow of water should be checked at the same time. Water coils must be cleaned regularly to minimize the danger of clogging. It is usually desirable to change the oil in these bearings at least once a year. Where the same oil is used summer and winter and the climate is severe, an oil with a minimum variation in viscosity with temperature change should be selected.

Horizontal bearings of the ring-oiling type are very reliable and require very little attention. The main thing is to be sure that the oil level is maintained at the proper point and that the oil rings are rotating with the shaft. The oil reservoir should be examined from time to time to make sure that it is free of foreign matter.

Ordinarily the same oil may be used for all turbine bearings. It should be a high-quality, straight mineral, medium-heavy oil with a viscosity of about 250 seconds Saybolt at 100 degrees Fahrenheit. A very effective method of keeping the oil in good condition is by means of a continuous by-pass system working in conjunction with a centrifugal oil separator. Where the oil cannot be treated by such a system, provision should be made for reconditioning the oil once a year or oftener. The oil should be allowed to stand idle for 10 days to settle out impurities, then filtered. It is not good practice to add more than 10% new make-up oil at one time to a batch of oil in service because of the danger of foaming.

18. Care of Hydroelectric Generators. Generators should be cleaned at frequent intervals to keep dust and foreign matter from accumulating on the windings or in the ventilating ducts. This applies with particular force to generators not provided with closed air recirculating systems. If the dust is not removed a firm deposit may build up and eventually clog the ventilating openings of the generator, causing undue heating or reduction of capacity. Various methods are available for removing the dust. Usually an air jet or strong suction is sufficient for the purpose. An air jet should be free from moisture and should not have too high a pressure behind it because of the possibility of injuring the insulation. Accessible parts can be cleaned with wiping rags. In some cases it may be practicable to surround the ventilating openings on the outside of the generator with canvas and to exhaust the air with a blower of sufficient capacity to produce a good flow of air through the ventilating openings within the generator. An air-pressure jet can then be used from the inside of the generator to loosen the dirt. By removing the dust to the outside of the building there is less danger of dust being recirculated through other generators in the station.

Oil should be kept from the generator windings at all costs. Oil not only is injurious to the insulation of the windings, but also it is an attraction for dust, which sticks and cakes until it is next to impossible to remove the

deposits. Because of the danger of unseen deterioration of the insulation and a ruinous short circuit later, no oil leakage should ever be permitted to continue and every effort should be made to see that oil vapors do not enter the air stream circulating through the generator windings. Megger tests of the generator windings should be made periodically.

19. Care of Governors. A governor consists of a rather delicate, intricate train of linkages and valves. If any one part in the system of linkages and valves fails to function properly the whole governor fails in its purpose. No item of power-plant equipment is so quickly and vitally affected by dirt accumulation and insufficient lubrication as the governor. Lodging of a particle of dirt in the pilot valve causing it to stick is a typical and perhaps the most common source of trouble. Binding in some of the bearings due to lack of lubrication, gumming of the lubricant, or dirt accumulation is another frequent cause of trouble.

The importance of keeping every part of the governor clean and properly lubricated cannot be overemphasized. It should be wiped scrupulously clean each day with clean, lint-free rags. Lint might get into the vital parts of the governor and cause trouble. All bearings in the governor mechanism should be oiled at regular and frequent intervals to keep them working freely. A governor which is kept clean and well oiled will operate for many years without appreciable wear or change in performance.

The oil used in the governor system may well be the same high-quality, straight mineral oil that is used for the bearing lubrication system, but some manufacturers prefer a light-bodied oil with a viscosity of about 150 seconds Saybolt at 100 degrees Fahrenheit. The oil must be kept clean. Reconditioning of the entire oil charge by settling and filtering is advisable at least once a year, and oftener if there is much chance for dust or dirt to enter the system.

If the governor oil heats considerably, watch out for a leaking check valve or some similar condition causing an excess circulation of the oil. Periodic checks should be made of leakage by the amount of pumping that is required both with the unit on load but not regulating and with unit shut down. The check on leakage should include the main regulating cylinders to be sure not only that there will be an ample margin of power to close the gates in an emergency but also that the response of the gates to the governor pilot is prompt. The oil pumps must be watched for leakage of joints, temperature of bearings, and noise. Operation of automatic devices for starting and stopping the pumps and for preventing excess pressure should be watched [illegible].

Governors that are properly installed and completely adjusted during the initial period of operation seldom require any change of the primary adjustments thereafter. Many governors operate satisfactorily for years without any change of the pivot point or dashpot settings. On account of its delicate and involved mechanism, any adjustments of the governor that are necessary

should be made only by some one who is thoroughly familiar not only with the design and operation of the governor but also with the type of regulation needed for the system.

MAINTENANCE

20. Good Housekeeping. Maintenance of the plant and its equipment should be well organized before the plant is put into regular operation, and the first rule laid down should be one insisting on good housekeeping. There is nothing that indicates the character of maintenance better than good housekeeping. Invariably careless housekeeping and careless maintenance are synonymous. One accumulation of débris invites another. Likewise, a bright clean plant inspires attentive care for every piece of equipment. A clean plant is generally a safe plant. Good housekeeping can often be encouraged by assigning definite parts of the station to different shifts; this fixes responsibility and creates competition.

21. Inspection Schedules and Reports. No single item about the operation of hydroelectric plants is more important than inspections. In addition to the routine inspections by the operators, previously described, every plant should set up a rigid system of scheduling inspections and maintenance to serve a twofold purpose: first, to secure adequate maintenance; second, to record the history of individual pieces of equipment. For a large plant the system may well take the form of a master schedule giving the dates equipment is to be cleaned, inspected, or overhauled. From this master schedule, sheets may be made up and given to the maintenance foremen, to be returned at the end of the week with a report of the work done. A follow-up for unfinished items and a standardized method of recording the data chronologically for each individual piece of equipment would complete such a system.

Where a number of smaller plants are served by maintenance crews moving from plant to plant, the master schedule would originate with the division supervisor of maintenance and be prepared for the group of plants for which he is responsible. It is very desirable that all maintenance foremen keep diaries of their work.

Every plant should be given a thorough general inspection by a competent man at least once a year, preferably in low-water summer periods when weather conditions are favorable and the turbines may be shut down without loss of power output. Besides providing an historical record of conditions throughout the plant these independent inspections pick up many little details that would otherwise pass unnoticed. Such inspections should take particular note of leakage past the dam or any deterioration of materials that would affect the safety of the dam or other structures, and they should show the general level of housekeeping and maintenance in comparison with other plants of the system. Apparatus is inspected and methods of operation are reviewed. Notes are made on fire and accident prevention.

It is desirable that the results of the official inspection be tabulated on a standard form containing space for remarks on every part of the development. The tabulation might even include such minor items as amount of driftwood in the pool, condition of railings, roadways and painting. In fact, each report would be intended to cover a minute inspection of every feature pertaining to the successful operation and maintenance of the station.

22. Turbine Inspections and Maintenance. Long life and dependability are easy to attain if turbines are properly cared for and maintained. To be sure of such results it is important that inspections of the turbine pit be made frequently and on a regular schedule, supplementing these regular inspections with adequate and prompt maintenance. The old slogan "keep up maintenance" has a foundation in sound economies. Wear of turbine parts, particularly in the gate mechanism, is compounded; one thing leads to another, so that a major problem may be developed from a relatively minor lack of maintenance. A little correction at the start of a difficulty will be the cheapest defense against serious and perhaps permanent loss.

As a trouble indicator, it is desirable that dial indicator readings be taken, preferably daily, to determine the oscillations of the turbine shaft. If these readings are taken from fixed locations and mountings and recorded, they give an excellent indication not only of wear in the main turbine bearings but also of foreign material lodged in the runner. The readings taken before and after bearing adjustments are of particular value.

In connection with the maintenance of turbines particularly, it is important to keep an adequate apparatus record Apparatus cards for the turbine should show the dates and results of inspections, adjustments, and repairs. The various clearances taken during a turbine inspection are usually recorded on a regular form. These records are very important because in many cases wear and changes are too gradual to be otherwise detected It is only by study of these apparatus cards that the causes of some of the difficulties and their remedy will be apparent.

In preparing for work in the wheel pit certain precautions should be taken for the protection of the men. An ample supply of fresh air is essential, particularly in warm weather. In cold weather it should be heated to moderate the wheel-pit temperature. Ventilation must be adequate when considerable oxyacetylene burning is required. Where necessary, foul odors from decaying vegetable matter can be relieved by steaming the wheel pit for several hours before the men enter. It is desirable that air-driven tools and 32-volt lighting be used in wheel-pit work if at all possible. Two sources of lights should be provided. When a wheel pit is unwatered during freezing weather, care should be taken not to allow the air temperature in the vicinity of the turbines to fall below freezing. Occasionally water or mud inside the hollow turbine gates has frozen and exploded the gates. Other parts of the turbine or appurtenances can be severely damaged or wrenched by the freezing of trapped water.

Without attempting to give a complete outline, the following are some of the steps to be taken in making turbine inspections. It is assumed that the inspector has provided himself with sets of thickness gages and the indicators necessary to measure the various clearances properly, that he has at hand any memoranda recorded during previous inspections, and that he has made himself thoroughly familiar with the history of the particular unit being inspected. Procedure might be about as follows:

(*a*) Clean the water passages of any driftwood, stones, or other débris that may have been lodged inside. Install the necessary facilities for making the inspection conveniently. Clean the inside of the scroll and other accessible parts in the water passages periodically to remove deposits of bacterial growth, etc. The authors know of one plant where 650 to 950 hp was restored to each 54,000-hp unit of the plant by scraping the surfaces with a steel hoe. The bacterial growth at time of removal had accumulated over a period of 18 years, and the cost of its removal averaged about $300 per unit in 1947. Scraping with steel hoes or glass is more effective than sand blasting.

(*b*) Closely examine the runner and guide-vane surfaces for pitting. This will include in particular the top and bottom sides of buckets near the outer band, also the runner discharge ring, and the top, bottom, and sides of the guide vanes where the seals occur. If pitting is occurring it will deserve special notation and study along the lines indicated under the next paragraph heading. Grease the guide vanes in the usual manner, and note how the grease comes out of the top and bottom bearings.

(*c*) Examine the gates for signs of wear, breakage, or bending of the thin vane edges due to foreign material. Make sure of sufficient clearance top and bottom to assure freedom of movement. Note condition of gate-stem packing glands, and renew packing if necessary. Examine the gate links for wear, the bearings on the shifting ring, on the connecting rod, and on the regulating shaft to see if any are due for replacement. Record the clearances of all bearings.

(*d*) With the aid of thickness gages, measure and record the runner clearances around the periphery of the runner band at each quarter point, also at the corresponding locations on the runner crown.

(*e*) Measure and record the clearance between the steady bearing and the shaft on the quarter points corresponding to the locations used for recording turbine-runner clearances.

(*f*) Study all the recorded clearances in their relation to those previously recorded. Where adjustment is possible, take up the bearings to correct for any appreciable wear that has occurred.

(*g*) Close the guide vanes tight, and note leakage areas. If any turbine gates have become bent or broken they should be straightened and repaired. Where scale has accumulated along the closing edges of the gates they should be scraped clean. With the aid of thickness gages measure and record the leakage clearances at the top, middle and bottom of the vanes. Closure of the

gates should be uniform over the full length of the gate, and all should be reasonably tight.

If any of the gates stand open, closure usually can be accomplished by adjustment of the eccentric pins in the gate linkages. Where no adjustments are provided, eccentric pins or special links may be machined and installed for the individual gates needing change. If it is impossible to adjust one of the gates for closure even with the maximum adjustment of its eccentric pin, a complete readjustment of all eccentric pins will be required. To do this, first set all eccentric pins to give maximum opening of the gates. Then close the gates carefully with the regulating mechanism until some of the gates are seated. Starting with a gate on whose thick end closure is complete, adjust its eccentric pin so that the thin end of this gate seats on the thick end of the next gate. Adjust successive gates in the same manner, proceeding around the turbine in the direction of rotation of the runner until all gates are seated.

(*h*) On completion of the inspection and repairs, and just preliminary to the closing of the wheel pit, apply grease to all wearing surfaces, taking particular care to grease well those bearings that are accessible only when the wheel pit is unwatered.

(*i*) Open and close the wheel-pit drain valve a few times to make sure that it is in good operating condition and will not leak.

(*j*) Clear the water passages and approaches to the turbine of any débris accumulated during the inspection, and make sure that all tools and equipment are removed from the turbine pit.

(*k*) Make sure that the pit manhole gaskets and bolts are in good condition before closing the pit.

Where heavy parts are worn or broken, electric welding is likely to prove a low-cost means of repair. This is true particularly of badly worn stems and shaft. They can be turned down, then built up by welding with stainless steel, and turned to original dimensions. This usually results in a longer-lived job than the original. Building-up worn sections of pins or shafts by the metallizing process has been receiving considerable attention as a still lower-cost means of repairing such parts.

23. Turbine-runner Pitting. With the improved draft-tube efficiencies that have been obtained and with the tendency toward higher specific speeds, an increasing number of installations have been exposed to turbine-runner pitting. Greatest economy is usually obtained with the design of the runner crowded close to the point where voids may form on the surface of the runner and produce pitting. High static draft head, high water velocities over the surface of the runner, and too much curvature on these surfaces on the draft-tube side will increase the tendency toward pitting. While better knowledge of the causes of pitting may have resulted in designs closer to the point of pitting, knowledge of methods of combatting pitting have increased as well. For the story of pitting and its remedies, reference is made to two publications of the Hydraulic Power Committee of the National Electric Light Association: "Pitting of Hydraulic Turbine Runners," April 1926, and "Welding of Water

Wheels," April 1931; also to a McGraw prize paper, "Prevention of Pitting Caused by Cavitation in Water Wheels," by T. C. Stabley, in the *N.E.L.A. Bulletin* for September 1932; and to an excellent paper by James M. Mousson, "Practical Aspects of Cavitation and Pitting," *Edison Electric Institute Bulletin*, September and October 1937.

Frequent inspections should be made during the early stages of operation; if pitting shows up in any spot, careful records should be taken. Runner buckets should be permanently numbered for identification of notes recording the progress of pitting. Many times it is possible to ascertain from the station records the type of operation which produces the pitting and thereby to learn how to avoid a major share of this difficulty. Operation of a turbine at the gate opening required for the last kilowatt of maximum capacity may not be justified where pitting is a factor. Extended periods of operation at high gate openings or operation at very low tailwater, particularly during hot weather, may be the times when pitting is most active on the draft-tube side of the runner. The best means of whipping this problem is to keep a complete, accurate history of the progress of pitting and to start a program of testing methods of repair or changes of design before pitting has progressed to the point of being serious.

In the majority of cases repairs by welding have proved the most effective means of restoring a unit to its original condition. For light pitting ¼ to ½ in. deep over limited areas on cast-steel runners, one of the most durable surfaces may be secured by chipping the pitted area down to clean metal and building back up to a full contour by electric welding, using 18–8 stainless steel (18% chromium, 8% nickel). Care should be taken to distribute the heat of welding as much as possible. Peening each layer helps relieve temperature stresses due to cooling. The welded areas should be ground to a smooth contour in places where the rough surface might induce pitting beyond the patches. For successful welding in an overhead position with 18-8 stainless steel the alloy should contain less than 0.07% carbon. For more deeply pitted areas it may be advisable to weld up to within two beads of the finished surface with a low-carbon steel, then to use one bead of straight 18% chromium steel, finishing off with the final surface of the 18–8 chrome-nickel steel. This both reduces the cost of welding and results in a better bond and a more uniform gradation of properties from the parent metal to the surface. Where large masses of welding are involved, particularly on cast-iron runners, steel studs into the parent metal are desirable to provide a better bond, particularly against the differential stresses due to temperatures.

Some wheels have been successfully repaired by inserting stainless-steel plates over the pitting zones. Other methods, such as copper or bronze plates, metals deposited by the metallizing process, metallic cements, and rubber have been used with varying degrees of success. The possibilities of success with any one method have by no means been exhausted. Each case of pitting is a study in itself. Pitting has sometimes been due to using a guide-vane bolt circle too small, with resultant overhang of the gates at rated capacity.

Pitting from this cause has been reduced by streamlining the overhanging edge of the guide vanes on the pressure side.

Pitting of turbine runners is not so serious a matter from the standpoint of cost of maintenance as is often supposed. It is not so costly a factor in water turbines as it is in steam turbines. Even in large plants where so-called "excessive" pitting has occurred, the cost of continuous maintenance has seldom exceeded $500 per unit per year or $9 per million kilowatts generated and has more often been about half of this amount. In one plant having fifteen 16-ft-diameter wheels in the "moderate" pitting class, the cost of pitting repairs over a 24-year period has averaged less than $80 per unit per year and less than $2 per million kilowatt-hours generated.

24. Generator Maintenance. Some of the items to cover in the regularly scheduled generator inspections are given below.

(*a*) Examine armature insulation for any mechanical damage, for loose taping, and for the necessity of revarnishing. Check cording of armature coils for tightness. Check for loose or missing wedges in the armature slots. Watch for any evidence of oil leakage into the windings.

(*b*) Inspect for loose laminations, and if any are found wedge them tight. Rust spots will usually indicate where the laminations are loose.

(*c*) Check for tightness of the bolts clamping the laminated structure of the rotor. Make sure that the locking devices are in good condition.

(*d*) Check tightness of parts attaching field poles to the rotor rim. In some cases this will involve testing the tightness of bolts and in others the tightness of taper keys.

(*e*) Check the interconnections between field poles, the insulating collars at the outer edges of the field pole coils, the condition of the leads between the field winding and the collector rings, examining particularly their insulated supports for tightness. Check the collector rings for surface condition and for eccentricity. Replace any short collector-ring brushes that might induce a flashover.

(*f*) Check air brakes to be sure that they release properly. Check brake shoes for wear.

(*g*) Record clearances between rotor and stator with the rotor in four different positions.

(*h*) Upon completion of any work on a generator, a careful inspection should be made to be sure no tools or materials are left in the machine. A small piece of metal working into the air gap may be the cause of serious damage.

25. Governor Maintenance. A thorough annual inspection of the governor mechanism is the best way of insuring satisfactory performance. The governor should be completely dismantled by a man trained for that particular work.

Pivots and bearings of the flyball mechanism should be inspected for wear. All ball bearings should be carefully cleaned and immediately flushed with hot oil. The dashpot piston and cylinder should be cleaned and tested to see

that no friction is present. Friction in the dashpot will seriously affect action of the governor. The pilot valve and its bushing should be checked for wear and if excessive both should be renewed. The bearings of the restoring mechanism should be checked for lost motion, and, if appreciable, the lost motion should be removed; otherwise hunting may occur. Lost motion and friction are the enemies of good performance.

The mechanism driving the flyballs should be inspected to make sure that there will be no periodic vibration coming to the pilot valve to cause erratic governor action. In the case of gear-driven flyballs, backlash in the gears due to wear should be watched. In the case of belt-driven flyballs, the belt should be endless and the splices smooth.

26. Trash-rack Maintenance. Keeping the trash racks of a plant clean is one of the major problems of the operator. Leaves, drift of all kinds, and ice may cause concern. In the colder climates, frazil ice may be one of the worst offenders and can be the most serious. Ice crystals may attach themselves to the trash-rack bars so rapidly as to completely shut off the water from a unit within a couple of hours. If this happens, collapse of some of the racks is probable. In some plants loss of service units on account of frazil ice would shut down the plant. Low velocities of approach in the forebay and deep intakes tend to lessen troubles from frazil ice.

Anticipating the conditions favorable for the formation of frazil ice and detecting its initial formation is important. A water thermometer and a frayed rope or chain indicator in the water ahead of the trash racks usually will give sufficient warning to get a maintenance crew on the job and remove the top racks in time to prevent curtailment of plant output. When this is done, the possibility of trash entering the wheel pit must be reckoned with. Where the water passages in the runners are small, a station may have difficulty with frazil ice choking the runners themselves, and a complete shutdown of the affected unit may be necessary for a period long enough to drain the pit and melt the ice by steaming the wheel pit. Means of heating the trash-rack bars by steam or electricity have been successful in some plants in permanently eliminating frazil-ice troubles.

Many plants are provided with mechanical rakes which are particularly useful for the smaller rack bar spacings and also fairly effective for some of the heavier material. In plants where heavy waterlogged timber, tree stumps, and the like form the major portion of the débris stopped by the trash racks, special grappling devices have proved effective. It is seldom necessary to resort to a diver to remove débris, although in some types of plants this may be required periodically. In some instances it may be helpful regularly to check the loss of head between some point ahead of the trash racks and some point in the penstock near the entrance. Considerable reduction of head may occur before the effect of drift accumulation becomes noticeable on the surface.

In streams carrying heavy drift, a floating log boom diagonally across the current in the forebay is effective in diverting the major share of the débris

to the log sluice or spillways over which the drift may be passed at intervals. Curtain walls in front of the trash racks also help in catching drift before it reaches the racks. In some plants the major part of the drift is collected along the smooth curtain wall in front of the power-plant intakes and is hauled to the log sluice by means of winch and drag line.

27. Maintenance of Structures. In climates subject to frequent freezing and thawing, maintenance of concrete structures may present a few problems. Surface disintegration of concrete occurs when three factors are present in combination: (1) porous concrete, or porous aggregate in the concrete; (2) water in the concrete; (3) and freezing. Restrict any one of these factors, and disintegration will be a very slow process. A well-proportioned dense concrete of 1:2:4 mix may never disintegrate in the presence of water and freezing. Even poor concrete saturated with water may suffer no disintegration if protected from freezing.

Every flat concrete surface exposed to the weather should be given a slope of at least 1 in. in 5 ft and should lead directly to drains that empty clear of the concrete. Water should not be allowed to pool up anywhere on exposed surfaces or allowed to dribble down vertical surfaces from drains or flat surfaces above. Exposed concrete surfaces should be kept clear of anything that will hold moisture and should be sealed against the entrance of water. Cracks that occur in flat surfaces should be channeled out and sealed with an elastic compound that will bond to the concrete and that will not harden with age or crack in winter. Expansion joints should be watched closely to make sure that water cannot enter.

In making repairs to disintegrated surfaces, for a permanent job it is usually necessary to provide an adequate mechanical bond with the old concrete. Also, there must be no opportunity for water to seep into or under the patchwork or come up from below by capillary action. For thin sections the use of Gunite concrete is satisfactory if the above precautions are taken. For heavier sections, good dense concrete adequately reinforced will be the best answer if water seepage into or through the old concrete is cut off. Reinforcing should never come closer than 2 in. to an exposed concrete surface.

For outdoor ungalvanized steel structures, one of the best paints is made of aluminum-bronze powder in a good-quality vehicle. Care in the preparation of the surface and in the application of the paint is essential for a durable job. Steel for an outdoor bus structure with ordinary exposure painted in 1925 with one shop coat of red lead and two brushed coats of Pittsburgh standard varnish aluminum bronze powder in No. 20 vehicle was still in good condition 13 years later. Only a few exposed sharp corners near the base of the columns had shown any signs of rust. Where a lead-base paint is desired for outside steel, three coats should be used, each succeeding coat using a smaller percentage of drier. The first coat should be harder and less elastic than the last so that when the outer coat hardens on exposure the film will be of a more uniform texture. The formula of the Metropolitan District

Commission of Massachusetts for red lead paint is a good one to use. (See Publication D15, Edison Electric Institute.)

For best results with a red lead paint on steel that is to be submerged, each successive coat should be harder than the preceding one, to resist the softening action of the water. This gradation may be accomplished by adding varying percentages of finely powdered litharge to the red lead-linseed oil base. For locations like the inside of penstocks where velocities may be high, the manufacturers may recommend percentages of litharge by weight ranging from 6% for the first coat up to 10% for the third and last coat. For steel in quiet water the recommended percentages may range from nothing in the first coat up to 6% in the third coat.

Hot applications of bituminous paints have been used for underwater steel with varying degrees of success. Biturine and Bitumastic paints both have their advocates. In *Electrical World* for Aug. 18, 1934, J. S. McNair reported the successful use of Biturine for the inside of penstocks. One job was reported in perfect condition after 7 years. He stressed the necessity of having a rough, but clean, sand-blasted surface to obtain a good bond.

Few of the many paints and materials designed to prevent corrosion of underwater steel last more than 3 or 4 years, and the expense of trying to keep the steel in its original condition may easily run into considerable money. In places where the steel can be replaced readily and where failure of the steel would not involve human life or the safety of other structures, consideration should be given to letting nature take its course after the first paint job. It may be better to let the steel develop whatever protective coating it can and then to replace the steel at the end of its useful life, rather than to keep cleaning and painting the steel at regular intervals. This statement applies with particular force to trash-rack bars, skin plates of crest gates, penstock interiors, and draft-tube liners.

28. Fire and Accident Prevention. Provision of adequate equipment for the prevention of accidents and training in first aid and artificial resuscitation have so long been the practice of operating companies that it should hardly be necessary to emphasize their importance again. The National Safety Council Safe Practice pamphlets contain important recommendations covering inspection of equipment and handling of materials.

An adequate fire protection system is essential. Instructions should be given in the use of the proper extinguishers for the different types of fires, and fire drills and inspections should be conducted as a follow-up. In the matter of fire protection, it should be remembered that oil in any part of the power station represents a potential fire hazard. Oil should be stored where there is the least possible chance for a fire to start. Every effort should be made to keep inflammable material from accumulating.

Wood structures about the plant should be eliminated wherever possible in order to reduce fire hazards. Fireproof construction may cost more initially but be the most economical in the long run when its value in preventing fires and in improving the general appearance of the power station or its surround-

ings is taken into consideration. Keeping outdoor structures painted and the grounds about the power station neat and clean all go hand-in-hand with good maintenance of power-plant equipment and fire prevention.

Hydraulic Operation

29. Duties of Operating Engineer. As distinguished from operation of equipment, hydraulic operation or the planning part of system operation calls for the analytical research type of mind. Hydraulic operation usually covers responsibility for the forecasting of river flow, daily power available, hydraulic-plant loadings for best efficiency, control of river levels, routing of floods, use of pond storage, and handling the load of such customers as are supplied secondary power. The work of hydraulic operation may extend from the plants themselves to the central load-dispatching office, where load allocation plans for the hydro plants would be supplied to the load dispatcher.

Systems with several plants and storage capacity within a single watershed may provide a hydraulic load dispatcher whose duty it is to see that storage waters and plant outputs are regulated so as to meet the needs of the system in the most economical manner. The hydraulic load dispatcher's report to a hydraulic engineer, who acts in a supervisory capacity, is responsible for the general plan of hydraulic operation, the analysis of records, the preparation of long-range forecasts, plans for best use of storage, plant efficiency schedules, river-level control plans, and all matters relating to the most economical use of hydro on the system. All plans for hydraulic operation are executed through the load dispatcher.

One large system with two major hydroelectric plants provides an engineering control paralleling the operating organization. This control supplies the general system plan for coordinating hydro with steam operation. It also provides the follow-up for efficient operation in the hydro plants. This independent engineering control works closely with every phase of system operation. Plant engineers are responsible for the efficient operation of their plants in cooperation with the operators. The plant engineers secure the hydraulic statistics for their respective watersheds and pass them on to the central office.

Problems of hydraulic operation naturally divide into two groups, one concerned with plant problems, the other with system economy in the use of hydro. The plant problems of hydraulic operation will be considered first.

30. Control of River Levels. Each hydro plant has certain obligations which must be met in connection with regulation of stream flow. Usually some state or federal agency will specify a minimum flow to be maintained below the dam in the interest either of water supply, stream pollution, or conservation of fish. Usually this will be the lowest natural flow that occurred before the dam was built. On some streams the daily range of output may be limited by requirements of navigation below the dam. In run-of-river projects where flowage lands are purchased up to levels corresponding to a

fixed elevation at the dam, there is the problem of operating as closely as possible to the upper limit to gain on head. In low-head run-of-river plants, the control of river levels by regulation of the load on the plant involves a delicate problem of load dispatching if maximum head and output are to be obtained.

When regulating gates are open on the dam, close control of river levels is more readily obtained. At such times fairly uniform flow can be obtained by varying the discharge through the spillway in synchronism with variations of plant load. This is important during floods when care must be taken not to exceed the levels which would occur under natural conditions. Though the type of spillway control provided will usually be determined by other considerations, it is well to remember that dams with fixed crests and no regulating gates have a distinct advantage from the standpoint of flood control because they do not involve artificial control of river levels.

31. Spillway Operation. Most of the recent hydro plants have spillway aprons designed to minimize erosion below the dam. Although it does not furnish a complete story, model-testing has supplied many valuable data. Various means of destroying velocity have been evolved, and they have proved effective in reducing the erosive action of spillage. There are, however, few plants whose foundations are such that they do not have to continually watch erosion below the spillways. It is desirable to operate spillway gates according to a fixed schedule prepared in advance. This schedule might be determined in part from full project model tests and in part by observations in the field. It may be possible to distribute the flow in small amounts between alternate gates across the spillway section and to produce little disturbance as compared with putting all this flow through one or two gates. In this connection it is well also to watch out for wide areas in the flood channel below the dam where sweeping eddies can form and augment the flow and scouring action in the main channel below the spillways.

Sometimes it is possible to concentrate the flow over the spillways in locations where the least scour will occur, or in locations where, if scour occurs, the damage may be repaired at the least expense. The effect of various combinations of gate operations on tailwater levels and plant output should also be considered. In connection with spillway operation, it is highly important that soundings below the spillways be taken after each major period of waste. Similarly, it is important that the contour of banks subject to erosion be taken often enough to keep a record of progressive changes due to spillway discharge.

In general, it will be found cheaper in the long run to experiment with frequent small expenditures for channel correction and bank protection than to spend major sums for so-called permanent works. In this connection, constant watchfulness and knowledge of what is going on is all-important. A very minor amount of stream-flow correction or bank protection at the outset of impending trouble will very often accomplish the same result as a major expenditure later on.

32. Routing of Floods. This is one of the delicate problems that has to be dealt with at dams where flood flows are controlled by crest gates. On streams having a wide cultivated flood plain this problem requires skill and care. For proper handling, prompt reports of rainfall are needed from stations close to the reservoir, and they must be interpreted quickly into a forecast of stream flow. By active forecasting, it is sometimes possible to anticipate some of the lesser crest flows in advance of their occurrence and to modify the flood crest somewhat for the benefit of people downstream. Lacking these reports and forecasts, pond stages at the dam must be relied upon to measure the increase or decrease of inflow to the reservoir. Wind may affect the pond-level readings to such an extent that it is difficult to know exactly what the natural elevation of the pond is, and there may be considerable lag between the actual flow and the regulated flow past the dam.

Contrary to views frequently expressed, it is seldom possible at most dams actually to draw down a reservoir ahead of the crest of really big floods. The big floods which do the damage occur as a result of excessive rains falling on ground filled to capacity, with surface depressions and small ponds all filled, and with river channels and reservoirs also full. Under these conditions, with most projects it is very difficult to accomplish anything in the way of reducing flood crests.

33. Plant Economy. In order to close the broad gap between engineering test data and its effective use in daily operation, the plant engineer and the operators must work together. The plant engineer should have a detailed knowledge of all the technical factors affecting hydraulic efficiency. This technical background must be adapted to the needs of the operators through the medium of simple plant operating curves and efficiency schedules. Efficiency operation should at all times be the second consideration of the operators, their first and primary duty being that of maintenance of service. Ordinarily the plant operators should not be burdened with engineering details that might detract from their effectiveness as operators. Nevertheless the operators should have a good working knowledge of the points where water losses are serious.

There are any number of opportunities for economy within a plant after the loading schedule has been assigned. It is only the plant operators who can keep the units balanced on the load at their most efficient point for the load assigned to them. Watchfulness on the part of the operators in handling their machines yields economies directly proportional to the efforts they make. A careless operator who allows his machines to wander—one high and one low, because it is the easy way—can waste several times his salary in water losses. On the other hand, with such tools as good sensitive governor equipment, gate-opening indicators at the switchboard, and a sensitive recorder of system frequency, a good operator can attain results very closely approaching perfection.

The operators should make every effort to reduce the time required for starting the machines and be on the alert to take machines off the load

promptly. There is always a tendency after a machine is once on the load to let it ride a little while after the load falls off to be sure that the load will not swing up again. This is a point where close watchfulness of the operator is essential for best plant economy. It is also the place where a follow-up by the plant engineer is important.

Some plants have been provided with equipment for loading machines at their most economical point automatically, and unusually good results have been attained thereby. Automatic loading of units for best efficiency is one way of applying the engineering test data to actual operation and is most useful in connection with the smaller plants where the operating force is small. In a large plant the same results can be attained by manual control with the further advantage of coordinating station operation with the system plan. With manual control there is less danger of the operating procedure's becoming crystallized and less danger of system economies' being overlooked. By working closely with the operators, the engineers gain the operating viewpoint that is as essential to efficient hydraulic operation as is the engineering viewpoint to the operator.

The plant engineer is usually made responsible for checking the hour-by-hour economies within the station. He should review operation each day and call the attention of the operators to any spots where inefficient operation may have occurred. Compliments should be given where the operating record shows close attention to efficiency. Good operation invariably results where someone with a detailed knowledge of efficiencies obtained can recognize the work done and attach that record to the man who made it. If the best operator working a load schedule day after day fails to have his attention called to oversights, or if his consistent efforts to attain good results are not recognized, he will soon slip into the habit of operating his station with a margin of safety larger than necessary, even though this margin may cost the system a considerable amount. Such losses are easy to overlook.

34. Watching Turbine-gate Openings. One of the most fruitful sources of plant economy comes from operating hydro units as close as possible to their point of maximum efficiency. There is a vast difference between maximum wheel efficiency and the average efficiency of normal production. The efficiency of a hydroelectric unit is usually greatest when it is turning out around 0.80 of its full gate capacity. Many units have very flat efficiency curves such that the load may be varied from perhaps 0.60 gate opening to 0.90 gate opening without sacrifice of more than 2% of peak efficiency. Beyond this range of gate opening, however, efficiency falls off very rapidly, particularly as compared to the same data for steam units.

It is not possible, with changing load conditions, for all plants to operate at their point of best efficiency. Hence, the hydro plant will have to be assigned its share of load changes, and the loading of its units will depart from points of best efficiency. Each plant should have a machine loading schedule designed for obtaining maximum efficiency at any loading that may be assigned the plant by the load dispatcher. Close attention to efficiency sched-

ules in the matter of gate openings and the number of machines on the load will yield much in the way of plant economy. For excellent articles covering the matter of plant loadings for best economy, reference is made to a paper by E. B. Strowger appearing in the *National Electric Light Association Bulletin* for December 1929, and to a prize paper by S. O. Schamberger appearing in the *Edison Electric Institute Bulletin* for January 1935.

If the units in a plant are of different size or type, the most economical distribution of load among them must be determined by a study of their respective characteristics. Where a plant has units of the same size, it is almost invariably true that the most economical operation is obtained when all units are operating at the same gate opening, provided, however, that a minimum number of units are used on the load. An additional unit is not added until all the machines have passed over their peak of maximum efficiency to the point where the cost of carrying the load on the smaller number of machines equals the cost of carrying the load on the next larger number of machines.

In any plan for the division of load among machines, the loss of efficiency caused by governor regulation should be taken into consideration. Test efficiencies obtained under steady load conditions with the gates blocked are not realized when a governor lets the unit swing above and below the average load assigned to it. Too heavy a burden of regulation given to a unit for the short swings of system speed may cause a serious loss of efficiency that will not be compensated for elsewhere on the system. A badly regulating governor may cause loss not only on the machine it is controlling but also on other machines whose governors try to compensate for its swings.

Usually the load dispatcher requires some of the running reserve of the system to be carried on hydro. Some of this running reserve will be available at no extra cost from the overload capacity of the units above the actual loadings at which they normally run for best economy. The balance of the reserve must be secured by rolling additional units. Instead of letting the additional units share the load with the other units, it will usually be far cheaper to provide the reserve by motoring the additional units. These units would run with their scroll cases filled, their wicket gates closed, their draft tubes vented so that the runners clear the water, and with their governors set to pick up load quickly on a drop in speed.

35. Venting Turbines at Low Gate Openings. Occasionally system operation may call for operating turbines at low gate opening regardless of the poor efficiency. The units may be required at these low gate openings for use as synchronous condensers or as reserve for the system. There also may be periods during the bringing of a plant on load or of taking a plant off load, when units may operate at low gate opening for considerable periods of time. At low gate openings, a substantial water saving can be made on reaction turbines by the admission of air to the draft tube. Venting of reaction turbines below 0.30 or 0.40 gate will usually increase their output because it relieves their tendency to pump water at these low gates. Many plants have installed mechanically operated air valves for opening at a predetermined

gate opening, thus securing these savings automatically. Some of these air vents, however, have proved noisy, and there is a tendency on the part of the operators to cut them out of service on the least provocation.

The air-vent valves are also useful in breaking the vacuum in the draft tube promptly after the turbine is shut down, thus reducing the head producing leakage. The valves can also be used for breaking the vacuum when it is desired to motor a unit with the draft tube empty. In order to break the vacuum on a large turbine with a 15- to 20-ft throat diameter, a 4- to 6-in. air-vent valve will be required.

36. Operating Turbines at No Load. Where use of hydro units as synchronous condensers running at no load is called for, heavy water losses may occur unless care is exercised. Air should be admitted to the draft tube so that the wheel will run clear of the water. For example, in a plant having 15-ft-diameter water-wheel runners, one 9000-kw unit operated as a synchronous condenser requires only 500 kw from the system when motoring with the turbine gates closed and the runner clear of the water in the draft tube. On the other hand, if the unit is operated with water at speed-no-load the water use is so inefficient as to be the equivalent of 1500 kw in a normally operated machine. The same unit motoring with the gates closed but with the draft tube unvented requires 2000 kw from the system on account of the pumping action of the runner. Close watching is necessary to avoid the mistake of running the wheels in water. It is sometimes necessary to put air at low pressure into the draft tube to depress the water level sufficiently to clear the runner.

Where units are used solely as synchronous condensers and not for running reserve, turbine gate leakage may be saved by closing the head gates. If such is attempted, however, care should be taken that sufficient water is being continually supplied, by leakage or other means, to the bearings and also to the periphery of the runner to prevent it from heating if there is any danger of its rubbing in the clearances.

37. Leakage. Potential kilowatt-hours are lost whenever water gets by a plant without doing work. Leakage during periods of shutdown may represent a large share of the profit from the enterprise. It is important because it is continuous and is the maximum at times of lowest water when the plant may be generating the least. It is particularly important in plants designed for peak-load service where the developed capacity is large in proportion to the normal river flow. In such plants during low water many of the units are shut down for a good portion of the time.

The turbine gates are the most vulnerable points for leakage. When the unit is first installed, leakage through the gates may very properly be decidedly less than 1% of full gate discharge yet after a few years' operation the leakage may have increased to 2% of full gate discharge, especially if the gates have been allowed to get out of adjustment. During a low-water period, where 6 out of 10 units might be shut down, a 2% leakage on the individual units would mean a 5% reduction of plant output which in turn might easily

represent a 10% reduction of net earnings or net value of the project. Many cases have been cited in which gate leakage has been 4% or more on units with considerable wear and with closure of the gates poorly adjusted. Particular attention should be paid to minimizing wear of the guide-vane bearings and to keeping the guide vanes adjusted for tight closure.

High-head turbines particularly should be watched at the clearances between the guide-vane ends and the guide case, because, if there is sand or silt in the water, wear may be considerable in even a short time, resulting in excessive leakage when the unit is shut down. When there is no likelihood of a unit's being required for reserve, most of the shutdown leakage can be avoided by closing the head gates. Some form of tight valve near the turbine is particularly desirable on high-head units for closure during shutdown periods, not only to reduce leakage losses but also to decrease the wear at the sides and contact edges of the guide vanes.

Where flashboards are used on the dam, a daily patrol may be necessary to keep leakage down. Crest gates may have to be cindered after each period of spill. Though leakage over the dam can be an item of considerable importance if neglected, such leakage can be seen and it is much easier to control than the unseen leakage through the plant.

Periodic checks of leakage through the water wheels and through the plants are especially important. It is not sufficient to assume that there have been no changes. The checkup on leakage should include the unwatering valves, crest or sluice gates, log chutes, fishways, the dam and power-plant structures, and any other points where leakage might occur, particularly those places where it is not directly visible. Besides covering the matter of leakage such checks may be valuable in matters of the safety of plant structures.

Leakage past the plant may be checked by observing the tailrace with the station shut down tight but with the head gates open. Often wading measurements in the tailrace below the dam will determine the leakage accurately; or weir measurements may be practical.

Leakage through the individual units may be determined by the rate at which the water level falls in the penstock or scroll case with the head gates closed. This requires a determination of head-gate leakage, some work in computing penstock or scroll-case water-surface areas and the application of hydraulic theory, but it is possible by this method to obtain a fairly accurate measurement of gate leakage. Another method that has been successful in low-head plants is to close all head gates but one, and this one only partly, then to use current meters in the opening. Under these conditions the unmeasured leakage is negligible.

38. Wear and Deterioration as Affecting Efficiency. Turbine-gate leakage is quite different from leakage through the runner clearances. The latter leakage shows up in its effect on turbine efficiency. Many turbines run from 10 to 20 years without appreciable wear in these clearances and without appreciable change in efficiency. However, on high-head turbines having low specific speed the clearance spaces around the runner are very important as,

even with a new machine and 20 mils clearance, the leakage through this clearance area may affect the efficiency from 1 to 3%. With dirty water, this clearance may increase fairly rapidly to 3 or 4 times the original measurement, and the efficiency of the unit may be decreased from 3 to 10%. These clearances should be watched and definite limits set at which the wearing rings should be renewed. This applies principally to turbines operating at heads above 250 ft and more particularly to those operating above 400 ft. The clearances are not so important on the lower heads, and for units operating below 50 ft moderate clearances have very little effect on efficiency.

Pitting of the runner seldom has a serious effect upon the efficiency of the unit until the pitted area becomes so extensive and the pitting so deep that the shape of the water passage is materially changed. Before this occurs, however, it is usually found that the strength of the runner has been so seriously affected that it is desirable either to renew the runner entirely or to build up the pitted area by welding.

In some of the older plants deterioration of the apparatus may affect efficiency. Where a wheel case was built too light or where there has been movement of the substructure with temperature changes, distortion and loss of efficiency may have occurred. Broken gate seats, abraded concrete linings, excessive corrosion of rack bars, or other factors increasing friction in the water passages may lower efficiency to a limited extent.

39. Water Accounting Practice. In the larger stations where a full-time plant engineer is warranted, there should be a complete accounting of the water used in the plant each day. This daily water accounting should be complete from the inflow to the reservoir through evaporation, storage, plant use, and leakage. It should strike a daily balance with the change of flow below the dam as determined by discharge rating curves. The accounting should be designed to detect errors of meter readings and errors in the flow computations by comparing the computed change of flow with the change of flow indicated by the tailrace rating curve.

In many of the higher-head plants it is practicable to install continuous water-measuring devices, such as a Venturi meter, in the penstock, or a weir below the plant. Such devices are a valuable aid in the daily water accounting. They have more or less permanent ratings, and they give an independent means of checking use of water against power output. In the low-head plants where such devices are not possible, the daily water accounting should be supplemented by periodic checks of flow through the plant computed from half-hourly readings of gate opening. These periodic checks, which may be made perhaps every 2 or 3 weeks, serve as a running check of the plant operating factors used in the daily computations.

Where complete discharge tests have not been made, relative use of water may still be accounted for by the index method. This method utilizes pressure differentials as a measure of relative flow. The pressure differentials may be obtained from a change in penstock area, from a change in direction of flow, from the change between the impact and draft sides of the stay vanes,

or simply between the forebay level and a point inside the penstock or scroll case. The index method finds its chief value in extending ratings beyond the range of actual tests. If completely calibrated by water-wheel tests, the index method may provide a convenient means of supplementing the more tedious method of determining discharge from records of gate openings.

Present-day accounting procedure unfortunately does not attempt to measure the production efficiency of a hydro plant. This is because the fuel pile of the hydro plant does not represent an expense. The accounting reports will tell to a penny the production record of a steam plant yet have no record of carloads of fuel that might have been scuttled or saved at a hydro plant. To supplement the daily water accounting, a means of measuring the operating efficiency of hydro plants is definitely needed, yet not easy to obtain. Various utilization factors, or ratios of actual generation to that theoretically possible, have been used. Simple factors, such as over-all plant efficiency, kilowatt-hours per second-foot, and total water losses, all include the factor of operating efficiency but they do not differentiate between it and the fixed or built-in efficiency factors that are determined by the design of the plant, so that the real measure desired is not obtained.

An interesting attempt to account for the money value of all losses that occur in a hydro plant is described in an article entitled "The 'Balance Sheet' Idea Applied to Power Plant Operation," by G. R. Shepard, in a serial report of the Hydraulic Power Committee, *National Electric Light Association Publication* 278-34, March 1928. A more recent development of this idea is the use of a daily "hydraulic log" sheet designed to supplement the usual electrical log sheet. A description of the hydraulic log sheet was given in an article by E. B. Strowger in *Electrical World*, April 14, 1934.

40. Water-wheel Tests. The background for the daily accounting of water used in the plant should be adequate water-wheel and generator efficiency tests. The methods employed for making the water-wheel tests will vary from plant to plant. The different methods of measuring discharge are given in Section 22, Chapter 38. Operating experiences in connection with the metering of hydro-plant input are well described in the serial report of the Hydraulic Power Committee 1927–1928, "Water-wheel Testing and Operating Records of Plant Discharge," *National Electric Light Association Publication* 278-34.

For medium- and low-head plants, the water-wheel tests, to be complete, may require the use of a water rheostat to load the turbine artificially. The water rheostat allows the turbines to be run at variable speeds. Usually the head available during the period of testing is nearly constant, and only a fixed speed can be obtained from loads carried on the system. This will provide for tests of input and output for all gate openings at one head only. By testing a unit at variable speeds on a water rheostat at the available head, the actual performance of the wheel at the constant system speed and other heads can be computed accurately from the assumption that the efficiency of the turbine will be a constant for a given gate opening if the ratio of the speed

of the periphery of the wheel to the spouting velocity due to the head remains constant. Variable speed tests can sometimes be made by absorbing the energy on other units operated as motors.

All turbine ratings and generator ratings designed for use by the operators are preferably worked up in terms of the heads recorded by the plant gages and in terms of the gate openings or loads recorded on the meters. It is essential for the operating engineer to acquire a detailed knowledge of the derivation of these curves and an appreciation of all the basic factors that entered into their construction. Only by such detailed knowledge is it possible for him to be sensitive to the little constant losses that can creep into operation and continue day after day.

Where they can be done at reasonable expense, tests for efficiency should be made in every hydro plant at least once every 5 years. The tests do not need to be extensive but should cover the matter of leakage mainly. The ratings of the turbines are not likely to change much if no particular damage has been done to the runner and if gate areas are not disturbed. Changes will usually first show up as increased leakage.

41. Limitations Due to Heating of Electrical Equipment. There are usually several limitations on plant output which may govern at different times of the year. Transmission-line loads may limit the plant output under certain conditions; under others, transformer temperatures may be the limitation; or generator temperatures may govern. When such limitations affect the efficient loading of the hydro units, it is important that a thoroughgoing study be made of these ratings to see if they cannot be increased without sacrifice of safety. The rating of electrical equipment is merely a guarantee of a manufacturer as to performance; it is not necessarily the proper limit for the machine when put in operation. The operating limit is usually a temperature limit which is critical only during the hottest period of the year. During the cooler portions of the year when water and air temperatures are down, additional load can be carried over and above the summer rating, provided that conditions within the transformer or generator have been properly explored and are definitely known. Without this knowledge a larger factor of ignorance must be allowed for and the consequent losses absorbed on the system.

When heating limits the output of generators and in turn the efficient hydraulic loading of the units, the heating load can often be distributed between units in proportion to the temperature rise of each machine. Some units will heat up more than others, depending on the effectiveness of the cooling equipment, the accumulation of dirt, or possibly differences in the original construction of the units. Even units of the same design will vary widely. If a station is limited by the output of the generators, before operating at an inefficient hydraulic loading, all machines should be brought up to the same temperature limit by varying the power factor on the several units. In this way the heating load can be distributed among the generators to a large extent independ-

ently of the hydraulic or kilowatt loadings of the unit. In varying the power factor due consideration must be given to the requirements for electrical stability.

If the total load of all generators is limited by heating due to poor power factor, while there is an unused margin of efficient hydraulic capacity still available and needed, it will usually be found decidedly cheaper to carry some of the reactive load on another unit motoring with its draft tube empty rather than to operate the additional unit with water.

42. Use of Pondage. One of the most important problems that the engineer has in connection with hydraulic operation is the proper use of daily and weekly pond storage. On a large system there may be hydro plants without pondage that must operate run-of-river at 100% load factor unless the river flow is regulated by other plants with pondage upstream. There may be other plants with ample pondage which can operate economically at very low daily load factors, particularly during low water. Where pondage is sufficient, weekly schedules of drawdown are desirable. These weekly schedules will be governed principally by the amount of recovery possible between sometime Saturday and Monday morning. Figure 4 shows the distribution of generation by plants for a weekly load curve of a system served by two hydro plants, one a peak-load plant, the other a run-of-river plant. For the latter plant hourly pond levels are shown to illustrate the use of pondage.

A very common fault of operation is the drawing of the ponds during periods of peak demand and not permitting them to fill completely during the off-load period for fear of having to spill some water. This results in operation at heads lower than necessary. Careful analysis in many cases will show the desirability of operating so close to the upper limit of pond level that occasionally small amounts of water are spilled. The gain due to the higher average head will usually more than offset the loss due to the small amount of spill.

A careful analysis should be made to determine the maximum amount of economical drawdown for each dam. This depends largely on load conditions, pond capacity, and the total head. If the drawdown is to be great, the effect of loss of turbine efficiency, due to operating at a head different from that for which the turbine was designed, should not be overlooked in this computation. Where heads are lapped in a series of dams, a drawdown can be utilized without any loss of power. This lapping of heads has proved very advantageous in many cases. Charts should be prepared showing the kilowatt-hours in each pond for various drawdowns and the number of hours per day that each plant can be operated at any given load and any given river flow. The possession of such data is especially important and useful at times of minimum flow when every endeavor is made to have the hydroelectric plants carry as much of the upper portion of the load curve as possible. In connection with the use of pondage, it should be pointed out that there may be losses due to fluctuation of the load on a plant. With the fluc-

tuating load, more energy is generated during periods of decreased head and less energy during the period of increased head This effect is explained in some detail in Section 52

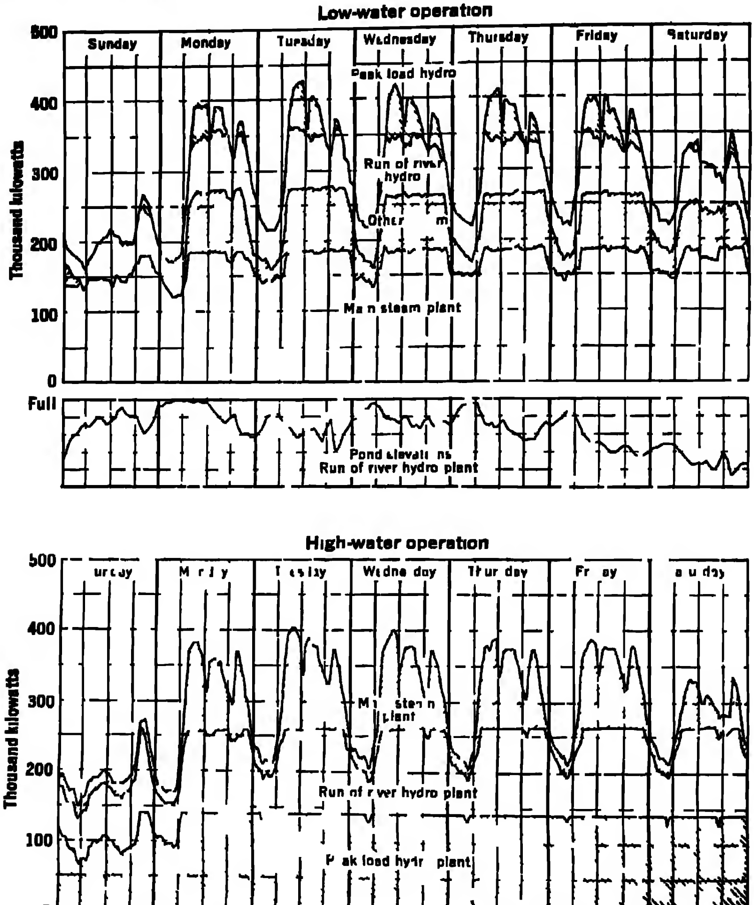

FIG. 4 Illustrating distribution of both run-of-river and peak-load hydro plant generation in a weekly system load curve

43. Forecasting Although forecasting is often a plant problem, it is here treated as a system problem, because where hydro plants are a part of a large interconnected system the forecasting of individual hydro plants is an essential part of the load allocation plan used by the load dispatcher

To be of value in forecasting, reports of rainfall and river flow must be prompt, and to be prompt, reliable telephone service in the basin is essential In many locations the system of timely reports of rainfall and river stages

provided by the U. S. Weather Bureau is invaluable. Even though their reports may not cover a drainage in sufficient detail for adequate plant forecasts, their timeliness forms a most desirable skeleton around which to build a forecast plan. River gages maintained by the U. S. Geological Survey and the U. S. Engineers are another very helpful aid. Arrangements can usually be made with their observers and with cooperative rainfall observers of the Weather Bureau for sending any special reports that may be required. It will usually be found necessary to establish additional rainfall gages, particularly in the areas closest to the plant.

It is desirable that each plant engineer collect most of the rainfall and runoff data for his watershed and for him to make his own estimate of runoff,

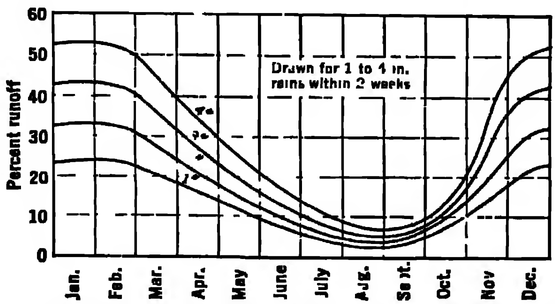

FIG. 5. Average seasonal variation of runoff percentages for a typical midwestern drainage basin.

transmitting them in turn to the central office. The plant engineer usually needs these forecasts for proper control of river levels at his plant. At the same time, on systems that have a large amount of seasonal storage, it is important that the central office make an independent forecast of runoff particularly from the long-range standpoint

Each drainage area has characteristics of its own. Each should have a statistical analysis made of past rainfall and runoff to determine average runoff percentages by months of the year Figure 5 shows a set of such curves for a midwestern drainage basin of 14,000 sq mi The curves are plotted for total rainfalls of 1 to 4 in within 2 weeks. By further statistical analysis, it is possible to develop average relations between rainfall and runoff for the different seasons of the year that will take into account the intensity of the rainfall and the condition of the soil—which condition can be approximately measured by weighting the amounts of rainfall in the preceding months. Two such relation curves are shown in Fig. 6 for the early spring and fall seasons.

These purely statistical analyses are helpful in keeping the forecaster conscious of the factors that will affect his estimate They have the effect of narrowing the band in which judgment must be exercised. To keep himself posted on the condition of the soil, the forecaster may keep a running record of cumulative rainfall by stations; with month-to-date, year-to-date, and

normals all shown. Other helps in picturing the absorptive capacity of the soil are the weekly weather and crop bulletins of the Weather Bureau which often include information on ground moisture. One or two small pilot drainages within the watershed, from which rainfall and runoff data can be obtained promptly by telephone or telegraph, are very helpful to the forecaster. Such drainages give a prompt and direct measure of per cent runoff which the forecaster can use coincident with the receipt of rainfall reports from the rest of the basin.

In the preparation of a forecast, it is essential that the influence of natural valley storage be considered in the integration of runoff from the tributaries.

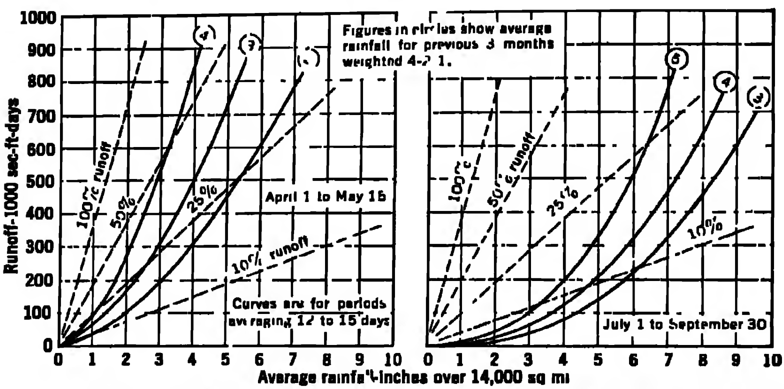

FIG. 6. Rainfall-runoff relations for early spring and fall seasons for a typical midwestern drainage.

This is the factor that is responsible for the flattening out of a flood as it moves downstream. The storage capacity of the ground itself, of the stream valleys, and of the plant reservoir must be kept in mind in the preparation of a forecast from a given rain. There is an enormous reservoir for absorbing rainfall in the ground. When this is filled and the stream beds are full, runoffs often approach 70% and sometimes exceed 80%.

Experience has shown that schedules for forecasting rainfall-runoff relations are valuable aids in developing judgment even though these schedules may be rough. They provide a common basis against which actual experience can be checked. It is desirable also that forecasts be recorded in some detail so that the forecaster and those who watch his work have some means of determining his average score. This is a very valuable tool in building and retaining experience.

In developing his estimate for use in allocating plant loadings, the central-office forecaster should be allowed time enough to prepare his forecast in an orderly manner and to exchange views with the plant engineers. In many cases, it is nearly as serious to overestimate runoff from a given rain as it is to underestimate. In the first case, too much low-cost energy will be dropped

from the steam plants; in the second, the pond may fill and water which might have been used through the hydro plant may actually be wasted. There are systems where a 5-hour advance warning that runoff will exceed plant capacity may mean a saving of as much as $1000.

For use in planning the loadings for the hydro plants and also for the steam plants, it is essential that the data from all watersheds and the load curves of the several plants for the preceding day be condensed into a form which one person can visualize. Such load curves and hydraulic statistics need be brought up to date only once in 24 hours, usually in the morning when most of the river stages and rainfall records come in. The data on loads may well take the form of weekly load curves plotted by hours or half-hours for each component part of the system with the generating station loads shown. A condensed weekly load curve is helpful because the days of the week show up in their true relation to each other.

With a condensed picture of the history and accomplishments of the previous day in mind and with all the morning's rainfall and river-stage records at hand, the most changeable part of the load allocation plan can be corralled. There remains the problem of fitting this hydraulic picture into the complete plan for the system, which, we will assume, is a major one including both steam and hydro plants.

System Use of Hydro

44. Economic Use of Hydro a System Problem. Interconnection of major generating plants over wide areas has emphasized the need of treating all generating efficiencies from the standpoint of the entire system rather than from that of the individual station Steam plant costs from hour to hour, outages of system equipment for maintenance, reserve for the system, line losses, and service reliability all enter into the problem of hydro-plant operation. Particularly is this true where reservoir storage on a system is large

System conditions are continually changing during the day. A hydro plant may be operating at perfect efficiency as a unit by itself yet be considerably out of balance with the best system plan. At one hour of the day the system object may be to obtain the maximum amount of capacity regardless of efficiency; at another hour of the same day the objective may be to save all possible kilowatt-hours, even sacrificing some efficiency in order to build up a reserve against more expensive steam generation a month or two later; at still another hour of the day efficiency of generation in the hydro plant may be the rule. And there are all shades of operation between these limits. For such a day the average efficiency of the hydro plant, as we ordinarily term efficiency, may be very poor, yet as a unit of the system its operation may be perfect.

There are so many chances for generating plants to wander from their best operating points that it takes much care to avoid waste. Operation is not a simple undertaking but is one of the most complicated, and within its sphere

there is a wealth of little hour-by-hour economies that can be discovered by painstaking engineering analysis. In the aggregate these little economies bulk large.

45. Interrelation between Hydro and Steam. On a large system, hydro can seldom be disassociated from steam. Since the energy cost of generation by hydro is so very small as compared to steam, all available hydro will be used first and the balance assigned to steam. To this extent hydro generation will control steam-plant loadings. Having available a considerable amount of seasonal storage roots the problem of economic system operation still more deeply in the hydraulic end.

The one who plans distribution of system generation not only must have a clear picture of the conditions affecting steam-plant operating costs, but he must also have a clear knowledge of all the factors that will affect the output and efficiency of the hydro plants. His viewpoint should be one developed from day-to-day experience with stream-flow behavior. He will need to picture distribution of rainfall, its effect on river flow, pond storage, and the stages that given load schedules will produce below the dam. He must understand the proper manipulation of reservoir levels to catch and use floodwaters in excess of plant capacity. He should be able to visualize all the spots where operating losses might occur and should be sensitive to all the little hydraulic economies as well as the big ones. Similarly, he must be familiar with steam-plant loading schedules for minimum cost, steam-plant increment costs from hour to hour, their requirements for outages for maintenance, and any temporary conditions that may cause departure from normal operating costs. He must from time to time compare notes with the plant engineers responsible for economies within their respective plants, whether steam or hydro.

46. Load Allocation Plans. Where several plants of both hydro and steam are involved there is need for well-grounded general plans for the allocation of plant loadings. These would include not only daily and weekly plans but also, where storage is available, a seasonal plan. These general allocation plans will outline the economic goals to be sought within the limits which are necessary for maintaining service. They may give the operating limits for hydro or they may designate limits of operation for the steam plant which in turn will control hydro-plant loadings. The load allocation plan should broadly define the range of operation of the plants but should not attempt to *fix* the operation of any plant. The daily-load allocation plan should be the guide for the assignment of hourly loads to the several plants.

The load allocation plan should be flexible enough to give each individual concerned in actual operation an opportunity to contribute the best of his experience. On large systems, where the economic balance between hydro and steam is continually changing, it takes the combined intelligence of many individuals to get the finest results. Attainments are in proportion to the cooperation among the several operators, the plant engineers, the system planning head and back around the circuit through the operators. "System mind-

edness" comes gradually but is hastened by good communication facilities. Rough spots in daily operation invariably can be attributed to lack of sufficient knowledge of the system or from the desire of an individual to work for the greatest economy of his own plant. It is not easy for a plant engineer to have his plant operated at poor efficiency for the sake of attaining system economy. It sometimes takes a blind faith to believe that the poor showing of his plant contributes to a good showing for the system.

Rigid schedules and too specific regulations do not secure the best results. Automatic devices can be considered aids but hardly substitutes for engineering in everyday operation. Too often elaborate schedules of operation are devised and gradually forgotten.

Good forecasting, a flexible system plan that will give the widest possible opportunity for intelligence to be applied all along the line, plus a follow-up are the essential elements for securing the best results.

47. Capacity Use of Hydro. Power available from hydro has two distinct values to a system: a capacity value and an energy value. Speaking in general terms, its *capacity* value depends primarily on how many kilowatts of the system peak it can carry with *minimum* river conditions. Its *energy* value depends primarily on how many kilowatt-hours of steam generated energy it can replace with *average* river conditions. Distinction between these two values will be more pronounced on systems having a large amount of seasonal storage available. In such systems capacity value will usually predominate.

Hydro will be used for maximum capacity when all plants of the system are taxed to their utmost to supply the system load, as will be the situation just before a new major unit of system capacity is brought in. When used for maximum capacity, storage reservoirs will seldom be drawn appreciably until the steam plants are operating at or close to their maximum output. By using reserve steam capacity in this manner when it is available, additional water can be held in storage for use during the period of actual outage of the largest unit, thereby supplying an additional amount of system capacity during the period of greatest need.

Drawdown of reservoir storage to leave room for catching floodwaters that might increase the energy output of the hydro plants is a secondary consideration when the system is short of capacity. The amount of the drawdown purely for catching excess floodwaters would be governed by the amount of excess waters that can always be counted on to occur.

On systems having a number of hydro plants, it is frequently possible, by careful analysis of operation, to demonstrate that the plants can dependably carry more firm capacity than they were originally assigned. As a system grows, the peaks of the load curve become sharper and the energy in a given peak cutoff decreases. This means that, with the same amount of energy available, a greater peak cutoff can be taken by hydro if enough storage is available to use the energy more and more in the peaks.

Where it was necessary in 1940, say, to operate a hydro plant at a 50% load factor during periods of minimum hydro, load conditions may have

changed so that it can be operated on a firm-peak cutoff basis at a load factor of 25%, thereby increasing its capacity value twofold. For example, consider a river which has a regulated minimum flow sufficient to carry a continuous load of 20,000 kw. If the prime load to be carried by the plant has a load factor of 50%, and if the necessary storage is available, then the firm capacity of the plant may be 40,000 kw. If the load factor is 25%, the plant might have a firm capacity of 80,000 kw. By confining use of the plant to a 3- or 4-hr daily peak with the extremely low load factor of 12½%, the plant theoretically might have a firm capacity rating of 160,000 kw. By using a small amount of hydro energy to carry the sharp irregularities of the top of the load curve, the firm capacity value of a given hydro system may often be substantially increased. The amount of this peaking up of hydro for firm capacity will, of course, depend largely on the amount of weekly and seasonal storage that is available.

48. Use of Hydro for Reserve Capacity. If properly handled, running reserve for the system, or synchronous condenser capacity for voltage control, can usually be provided from hydro at a much lower cost than from steam. On one system 30,000 kw of extra hydro reserve can be carried at a cost of $2 to $3 per hr while the same extra running reserve carried on steam would cost about double that amount. The reserve on hydro is obtained by motoring the units on the line with turbine gates closed, the runner clear of water in the draft tube, and the governor so set as to open up quickly on a drop in system speed. With proper governor equipment this is a very effective means of supplying system reserve. The allocation of this reserve to hydro plants has been the source of some gratifying gains in system economy. Unfortunately, however, it is necessary to guard constantly against the use of too much of this convenient reserve. Although its cost may be lower than steam there is a definite loss to the system if more reserve than necessary is provided in the load schedule or actually used by the operators on the load. Reserve should be watched just as closely on the steam plants as on hydro because it is the cost of the total reserve that counts.

In providing reserve capacity for *short* system outages, the leaning will be towards hydro on account of its quick availability as compared to steam. For running reserve capacity also the leaning will be toward hydro because it is cheaper. On the other hand, for the *long-continued* outages where quantity production is required, the emphasis will be on steam capacity.

Where there is ample storage, the capacity value of a hydro plant may be determined almost wholly by its ability to furnish the reserve to cover the outage of the largest system unit. This reserve has to be supplied by installed capacity somewhere on the system. Where such reserve can be supplied more cheaply by hydro than by steam, and if it replaces steam, the hydro can take full value as system capacity. In all questions of capacity values, especially where reserve is concerned, there is need to consider relative probabilities of the different types of outages, the probability of minimum hydro coming at

time of peak, and the probabilities of unusual load growth, all as related to actual coordinated operation on a specific system.

49. Energy Use of Hydro. Hydro will be used to yield its maximum energy value when there is an ample margin of capacity on the system. Securing a maximum energy value from hydro involves more than attaining maximum plant operating efficiency. It involves using the plant generation where it will reduce steam-plant operating costs the most. To accomplish this calls for an hour-by-hour analysis of the use of hydro in the system load curve.

To the extent which storage in the several hydro plants permits, the limited hydro generation during low-water periods will be fitted into the top of the load curve in order to relieve the steam plants of their highest cost generation. The exact proportion of hydro used in the top of the system load curve will depend on the difference between on-peak and off-peak steam increment costs; the amount of storage to be retained for long-range use; how many machines will be required for running reserve; how much provision must be made for outages for maintenance; and how close the hydro units will be operating to their points of maximum efficiency. Location of the plants with respect to load centers, and the capacity of tie lines, will also have a determining influence on the distribution of hydro energy in the load curve.

When hydro is flush it will for the most part be operated on base load, often with all units against their stop settings for maximum output. Under these conditions the steam plants would be operating at minimum load During the light-load periods in the early morning and all day Sunday, steam-plant loadings may be determined by the requirements for reserve and voltage control rather than by the amount of power available from hydro. When this is the case the steam units that are necessary would be running at their lowest practicable loadings, which may be about 20% of their rated capacity. During such periods increment steam costs would be very low, steam costs per kilowatt-hour very high, and the value of the last few hydro kilowatt-hours small.

50. Increment Costs of Generation. Since the value that can be attached to kilowatt-hours generated by hydro is measured by the steam-plant production costs they save, a clear knowledge of what these steam increment costs are for all types of loadings and for all hours of the day is essential to all determinations of the balance between hydro and steam

Figure 7 shows increment costs of steam generation for a hypothetical system supplied in part by hydro plants that have seasonal storage available. The curves show how increment costs might increase with the steam-plant loadings. It will be noted from the drawing that each unit coming on the load adds a block of constant losses which are a function of time only. Once the machine is on the load and carrying its minimum permissible load, increment costs from there on up are very nearly constant until the unit is pushed close to its maximum output. When a plant is brought on load there is a lump-sum cost to get the plant ready for service. Then, to operate the plant,

there is the fixed minimum hourly cost for operating labor plus the fixed cost of starting each unit

At the right-hand side of Fig 7 is shown an annual duration curve of load that might be typical for the same section of the hypothetical system It will be noted from the increment-cost curve that if all the load is to be carried on steam some very high-cost generation will result—the very top increment costs running perhaps three times the lowest At the same time, the duration of such generation would be quite limited The periods of low-cost steam

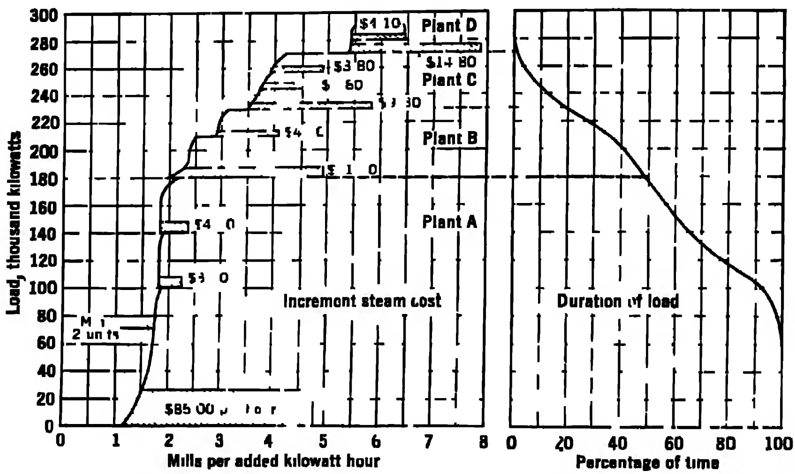

FIG 7 Increment costs of steam generation on a hypothetical system supplied in part by hydro A typical load-duration curve for such a system is shown at the right

generation are extended If all the available hydro generation can be used in the top of the load curve, the value of the water power will be large If all must be used in the base its value will be small

The object of any system operating plan would be to so fit hydro generation into the daily and weekly plan that a maximum amount of the high-cost steam generation could be saved These steam-plant costs saved may include the shut-down of an entire plant Where sufficient storage is available on a hydro system this use of hydro to avoid starting up inefficient steam plants is one of the main objectives to strive for in the system plan

51. Transmission Losses. In connection with the allocation of hydro loadings where the plants are supplying a load center over long transmission lines the economies to be effected on the hydro end may have to be modified to some extent by increased line losses The hydro plant and the transmission line should then be considered as a unit in figuring economic loadings

52. Dividing Load Fluctuations The theory that hydro is most useful in the top of the load curve operating at low load factor is true only up to a

certain point. Fluctuation of hydro loads and low-load-factor operation reduces the amount of generation which can be obtained from a given use of water. This effect is exaggerated in the low-head plants where the reductions in head caused by rising tailwater are a larger proportion of the total head. Increased fluctuation of load means less generation at the higher heads during the night and more generation at the lower heads during the day. The combination means less total generation from a given river flow than would be secured by uniform operation throughout the day. Such losses increase very nearly as the square of the kilowatt-hours transferred from the night to the day period.

To illustrate the point, consider a 50,000-kw plant operating with a river flow sufficient to carry 25,000 kw continuously with a full pond and a head of 30 ft. If, instead of operating the plant continuously, it is desired to operate at 50,000 kw for approximately 12 hrs during the day, leaving the plant shut down during the 12-hr night period, there will be a draft from storage of, say, 0.6 ft from pondage during the day and a storage of 0.6 ft during the night. During the period of generation the pond level will be an average of 0.3 ft lower than it would be for continuous operation at 25,000 kw. Similarly the tailrace will be higher during the period of generating at 50,000 kw. The average tailrace, for example, might be 2.0 ft higher The water available would therefore be used in the first case at a 30-ft head and in the second at an average head of only 27.7 ft. Total generation for the first case would be 600,000 kw-hr and for the second only 554,000 kw-hr, a reduction of 7.7%. Adding to this the cost of the extra starts and stops of machines and of their passage through inefficient zones of operation, the total loss will be around 9%.

Fluctuation of the load carried by a steam plant likewise increases unit production costs. Fluctuation may mean additional boiler starts and machine starts or it may mean carrying excess rolling or stand-by capacity during the light-load periods, all of which increases unit costs. Yet there is a practical limit to the gains to be made in steam-plant operation by improved load factor. Few steam plants are designed for operation at load factors above 80%. The last few increments of load-factor increase may yield very insignificant gains.

Disregarding some minor practical considerations, the broad statement may be made that all plants should share the load swings. Just how much of a share each plant should take, however, will vary from plant to plant. The proportion of the daily swing of load which each plant should take may be determined by evaluating the losses each plant would be subject to by reason of typical fluctuations. The results may be summarized in tabular form for each plant. The best division of the load swing will be that in which the total loss due to fluctuation of load on all plants is the minimum.

53. Stepping Hydro-plant Loadings. When we get down to the detailed load assignments from hour to hour in the different plants, advantage can be taken of varying steam loads within the efficient range of the turbine

and boilers on load. It is often possible to "step" the load of a hydro plant to avoid part-gate operation for any length of time. Each hydro plant has loading zones within which it can operate at top efficiency. Figure 2, referred to in Section 6, shows such zones for a six-unit plant. Instead of letting a hydro plant follow the changes of system load and pass gradually through inefficient loading zones, its load would be stepped promptly from one efficient zone to the next, provided the load change could be divided among the units of other plants without materially increasing their increment costs of generation. Recognition of this factor is particularly important where the units of a hydro plant are large. As pointed out elsewhere, operation at low gates on hydro involves big losses as compared to changes of economy within the usual range of ordinary operation on a steam turbine.

54. Long-range Uses of Storage. First in importance on systems where seasonal storage is available would be the planning of reservoir levels for the different periods of the year both to yield the most energy during low water for use in the top of the load curve and to catch excess flood waters for building up the total generation.

The many factors involved in such planning make it desirable to develop "rule curves" which will sum up experience equivalent to operating the system through all the river conditions of the past. These rule curves will show the economic drawdown of the reservoir before any regular spring or fall rains that may be characteristic of the drainage. They will also show how much margin it is desirable to leave below full reservoir level at all times of the year for absorbing casual floodwaters. The rule curves serve merely as a summary of a lot of detailed engineering analysis. They should not be looked on as rigid schedules but merely as guides.

Usually it will be necessary to develop two rule curves: the first for *capacity* operation where ability to carry peak is the main consideration; the second for *energy* operation where maximum economy is the guiding aim. The first rule curve will be less concerned with catching floodwaters and will aim to conserve storage primarily to yield maximum output during the extreme low-water periods; the second rule curve will utilize greater reservoir drawdowns to catch more floodwaters and generate the most energy. With these two rule curves available it is possible to judge fairly closely the proper position of the reservoir for any type of operation at any season of the year. A new plant coming on the system, a new transmission facility, or changes in the seasonal characteristics of the system load curve will alter these curves for system use of storage.

In northern climates, where ice affects the rivers considerably, there is usually a water shortage in the transition period between open water and ice-covered streams. During the initial freeze-up period, a considerable amount of water is stored under the ice cover in compensating for the lowered velocity of flow. This same storage will be released in the spring when the ice goes out. Reservoir storage may be used to tide over the temporary shortage at the time of the freeze-up. Where the breakup can be predicted with reason-

able closeness it may be possible to draw on storage and have it about used up just before the breakup and then to replenish the storage with the excess water that would otherwise be wasted. Careful analysis must be made, however, to determine that the plant will not be caught with reduced head, no storage, and a delayed breakup.

The next use of storage in order of importance would be in reducing the fluctuations of top average steam loads from week to week, thus in effect substituting low-cost generation for high-cost generation. Such use may often avoid the high cost of starting up another stand-by steam plant.

Seasonal storage may also be used in anticipating fuel shortages at the steam plants, and in connection with planned outages of major equipment. Many times, by planning ahead, sufficient storage may be conserved to handle such outages without loss. When the outages come they may be handled from storage without making it necessary to use inefficient generating capacity.

Judicious use of storage sometimes makes it possible and economical to shut down a plant entirely for extended periods and transfer the operating force to maintenance work on other parts of the system. This is a particularly fruitful source of system economy where the range of hydro operation is large and there are many small plants on the system.

55. Scheduling System Maintenance. The best periods for steam-unit outages for maintenance can be figured by the probability method from the records of stream flow in the various watersheds. Fitting steam maintenance into the periods of flush hydro and hydro maintenance into periods of low water will reduce maintenance costs very materially. An ideal combination would be to have steam and hydro plants close enough together so that joint shop facilities and joint maintenance organizations could be utilized and a more uniform load of maintenance work obtained thereby.

Outages of steam turbines for inspection and maintenance are often a serious tax on economic operation, particularly where the units are large and efficient. When they come down, their load must be supplied by other units which usually will carry higher top increment costs. With large units this may represent quite a spread in generating costs. Such outages can be very expensive. Outages for maintenance of efficient steam units during periods of low hydro may require generation on other steam units at double the cost.

On a system where this is the case, it may be quite advantageous to take steam-turbine outages ahead of their regular schedule if hydro conditions at the time are favorable. If hydro conditions are unfavorable and continue unfavorable it may be feasible to postpone the outages until hydro is favorable. It may be desirable to postpone the outage 6 months beyond the due date, waiting for an improvement in hydro. Usually hydro will get a break for the better within a 1-year period.

By scheduling steam maintenance in the above manner, the steam turbines may be serviced more often on the average. Though the direct cost for maintenance will be increased, the energy cost of the outages will be reduced far

more and the total cost resulting from maintenance will be substantially reduced.

56. Pooling Plant Capacities. Although it is highly desirable from the standpoint of operation for each load center or section of a system to have its own efficient generating capacity and its own reserve, it is also desirable that generation be pooled by means of adequate and reliable tie lines. Pooling of plant capacities will reduce the cost of reserve for the system and at the same time permit each plant to operate on the average much closer to its maximum efficiency point.

If all generating units can be operated tied together, the capacity of each above its point of maximum efficiency can be counted on as system reserve capacity in the event of a failure. With lightly loaded tie lines of adequate capacity, failure of the largest unit will be covered in part by the excess capacities of all the other machines of the system. Not only will less reserve be required on the section where the largest unit is located, but also the reserve provided for that section will be available to all the other sections. Contrasting with this, when the system is split up, each section must provide reserve for its largest unit, so that the total reserve provided for the system will be roughly proportional to the number of sections separated.

Pooling capacities will reduce the amount of inefficient part-gate operation. There is always a considerable loss due to the inability of a plant to follow a varying load curve without swinging the loadings of individual units away from their points of maximum efficiency. This loss is greatest when the load supplied is small relative to the size of the units. Generally speaking, tying two sections of a system together will halve the loss from part-gate operation.

There is, of course, the more obvious gain in being able to use units in different plants in the order of their efficiency when plant capacities are pooled. Such gains will be confined usually to the steam plants. In hydro plants the fuel supply cannot be diverted from one plant to another and must be used where and when it is available. Only where the pooling of capacities takes place in one station will it be possible to make gains from this source on hydro.

57. Service Records—Hydro and Steam. For a number of years successive Hydraulic Power Committees of the National Electric Light Association kept a careful record of outages of some 200 hydraulic turbines. Outages were classified according to causes. A summary of these data will be found in *N.E.L.A. publication* 216, April 1932. These records show an average "service demand availability factor" for the period 1924 to 1930 of 98.7% for hydroelectric units at the plant. "Service demand availability factor" was defined as the percentage of the time the unit should have been in service that it was actually operated. The total outage time of all units for the 7-year period was 4.3%.

Similar records have been kept for steam-turbine units by the Prime Movers Committee of the National Electric Light Association and of the Edison Electric Institute. A summary of these data for the period 1914 to 1936, inclusive, is contained in *Publication* E14 of the Edison Electric Institute. The

service demand availability factors given for the period 1929 to 1936 for about 300 steam-turbine units averaged 95.8%. The total outage time of all units for the same period was 8.5%.

Attention is being called to these data because of their usefulness in analyzing the operating records of the individual machines. They should not, however, be used as representative of the service records of plants, nor should they be used as a measure of reliability of service as between hydro and steam. Service reliability is a function not alone of the prime movers but also of the electrical layout—the bus, transformer, and transmission network design, the system of relaying, as well as the amount and location of the reserve carried in normal operation.

The low speed and simplicity of hydro units, their good service record, their availability within a matter of minutes for emergencies, all must be considered in conjunction with the transmission system and with questions of water supply, floods, strikes, tornadoes, and other contingencies, if a true comparison of hydro and steam plant reliability is to be obtained. The answer will be different for each system. Perhaps the only fair statement that can be made is that a given degree of reliability can be supplied more readily from a coordinated system of hydro and steam plants than from either hydro or steam alone. A combination of the two greatly decreases the chances of coincident outages.

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